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Featuring

**Report of the Committee on
Floods**



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REPORT OF THE COMMITTEE ON FLOODS, MARCH, 1930

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REPORT OF THE COMMITTEE ON FLOODS, MARCH, 1930

Chapter I. — Summary of Report

THE Flood Committee of the Boston Society of Civil Engineers was suggested by Ex-President Frank A. Marston and authorized by the Board of Government under the following vote:

At a meeting of the Board of Government of the Boston Society of Civil Engineers, held November 16, 1927, it was —

Voted, That the President be and hereby is authorized to appoint a "Flood" Committee, to collect, compile, analyze and report upon run-off of the New England flood of November, 1927, together with such other engineering data as may be deemed advisable.

The Committee has interpreted broadly its instructions from the Society to "collect, compile, analyze and report upon the run-off of the New England flood of November, 1927."

This report does not attempt to answer all the questions which have been raised, but it is intended to be a fair and scientific analysis of the November, 1927, flood, and the conditions which brought it about and resulted from it, and includes suggestions in the form of flood formulæ, preventive and protective measures which may be of service in the future.

The Committee has assumed that its investigations should cover, not only this flood but a comparison of it with other floods of the past, a discussion of future flood probability, flood formulæ, and flood prevention methods, with their cost and economic aspects. It was impossible for the Committee to study in detail specific rivers in New England, which would have required the expenditure of large funds. Matters have therefore been considered in general terms; but in so far as possible, the Committee makes definite recommendations and expresses an opinion of the general relation between the 1927 flood and other floods, and the questions pertaining to floods and flood prevention.

The report of the Committee comprises eight chapters. Following is a brief statement regarding their contents:

Chapter II — "*The 1927 Flood in New England.*" Gives a comprehensive statement of what took place during the 1927 flood. It was caused by excessive rainfall over most of the states of New Hampshire, Vermont, Massachusetts, Rhode Island and Connecticut, and a much less amount over western and southern Maine.

The chapter contains summaries of the rainfall and run-off accompanying the flood, and outlines the damage done. It points out that



PLATE I. — CONNECTICUT RIVER, HADLEY, MASS., SHOWING WATER SPREADING OVER TOBACCO FIELDS AND SURROUNDING TERRITORY. THE LONE STRUCTURE IS A TOBACCO BARN

the rainfall exceeded 5 inches over an area of about 22,030 square miles, or one-third the area of New England, and exceeded 7 inches over an area of 4,000 square miles, or 6 per cent. No rain gages were located on the mountain tops, but a study of the altitude-rainfall relations in Vermont indicates that the precipitation probably amounted to at least 10 to 12 inches in the higher elevations. Most of this heavy rainfall occurred within a period of 18 to 24 hours, and followed a month of more than normal rainfall, which had left the ground well saturated with water, thus paving the way for a high run-off.

There were two distinct flood areas, the larger from the Vermont-New York line to the east over the Green Mountains to the White Mountains, and from the Canadian line south to the Litchfield Hills of Connecticut, and a smaller one about central over the longitude of Worcester, Mass., and from the Massachusetts-New Hampshire line south to Long Island Sound.

From 3 to 7 inches of the rainfall reached the rivers over a large area, producing exceedingly high rates of run-off in many localities. The rates of run-off as compared with some based upon well-known formulæ are shown on Fig. 19. It will be noticed that several of them



PLATE II. — RAILROAD STATION, PROCTOR, VT., ON OTTER CREEK,
NOVEMBER, 1927

equaled or exceeded that shown by the Kuichling formula No. 2, for rare floods.

New England occupies a total area of 61,976 square miles; contained in 1920, 7,400,909 people; and had a total taxable wealth of \$22,552,000,000. The flood of 1927 took a toll of 87 lives, most of these in the Winooski and White River valleys of Vermont. The property losses for New England were about \$40,000,000, or two-tenths of one per cent of the total valuation, but of this, 26 million, or nearly two-thirds, was in Vermont. About sixty dams and abutments, all, however, of minor importance, were washed out. These were also mostly in Vermont and northwestern New Hampshire.

Railroads, highways, bridges, mills, farms and village property suffered; and besides the lives lost, many farmers lost their stock and

found their farms swept of soil or covered with sand and gravel by the velocity of the flooded streams.

Many of the river valleys in Vermont were submerged, transportation of all kinds was seriously interrupted, and the general scheme of life entirely altered for the time for much of the population in that state. The flood amounted to a national catastrophe. President Coolidge, as a native of Vermont, and Secretary Hoover, as the head of the relief furnished by the Federal government, made personal visits to the stricken neighborhoods. New England Council, Red Cross, all Federal and state departments, banks and other agencies, were called in to do their part. The use of aeroplanes to visit districts where all communications had been cut off furnished a new and most valuable aid.

Chapter III — "Previous Great Storms in New England." Gives the dates and a brief description of most of the outstanding storms of New England during the last two centuries. This list is not complete, as specific data on storms and floods previous to 1869 are rather meager, and much of the information previous to 1800 is almost in the nature of legend. This résumé of previous great storms has been included for the purpose of showing that floods are by no means new to New England, and in order to give a general idea of New England's flood history.

The histories of certain towns and villages located on the banks or within the flooded areas are full of references to previous floods, and some of them experience moderate freshets every few years; but when it is considered that the people occupying farms, houses, stores, and those in charge of highways, railroads and other structures, have profited by experience and "got above the high water," the 1927 flood must on certain rivers — the White, Winooski and perhaps others — be considered the very worst in the history of the State of Vermont.

Chapter IV — "Flood Factors for New England." Floods are caused primarily by precipitation which results in excessive run-off. The first part of this chapter gives a summary of such studies as have been made covering the great storms and excessive precipitation of New England. It includes the results of a study of excessive rates of precipitation at Chestnut Hill, a summary of the rainfall-area relations in six of the great storms of New England, and tables of rainfall probabilities in Maine.

There is usually a considerable difference between the amount of precipitation and the part of it that immediately runs off to produce floods. The second part of this chapter includes a study and discussion of rainfall and run-off relations.

Snow occasionally forms an important factor in New England floods. The third section of this chapter deals with this question. It is pointed out that melted snow is usually not an important factor on small and flashy drainage areas on which the concentration period is short and the peak flow reached within a few hours. On the larger and more sluggish drainage areas, melting snow occasionally becomes a most important factor. The addition of rain to this melting snow presents some of the most serious situations for floods on such drainage areas. An approximate relation is worked out between the mean temperature and the run-off from melting snow.

The last section of the fourth chapter deals with drainage area characteristics, showing why it is that frequently drainage areas of equal size are subject to floods of greatly different magnitude. The three most important factors are the shape of the drainage areas, the velocity of flow, and the amount of storage and pondage available to retard or absorb the flood. The relationships on any particular watershed are so involved that it is practically impossible to determine the effect of each of these factors analytically, but the various factors are all reflected in the flood hydrographs. It is believed that flood hydrographs afford the best basis for the study of the flood drainage area characteristics of a stream.

In comparing precipitation and run-off it must be kept in mind that the precipitation at the higher altitudes is greater than the relatively few gages indicate, and the run-off figures may be approximate for some of the gaging stations in operation at the time.

The number of rain gages, while relatively few for the 1927 flood, could be increased greatly at comparatively small expense; and this measure is strongly urged by the Committee.

Chapter V—“Flood Formulæ for New England.” In this chapter the Committee shows that a flood hydrograph once determined for a given river, even for an ordinary flood, will serve as a basis for the estimation of greater flood run-off, due to the fact that the base of the flood hydrograph (or time of flood period) appears to be approximately constant for different floods. This results in the equation—

$$Q = \frac{1290 R}{T} \quad \text{where}$$

R = the total flood run-off in inches of depth on the drainage area, and

T = the time in hours of the flood period

From this equation the Committee has developed a flood formula for New England in somewhat more convenient form for general use.

This proposed formula is similar to some of the other well-known flood formulæ, but with modifications making it more specifically applicable to floods of different magnitude. The formula is —

$$Q = C_F \sqrt{A} R \quad \text{where}$$

Q = the peak flow in cubic feet per second

C_F = a coefficient which is a measure of the drainage area characteristics of a stream.

$$C_F = \frac{1290 \sqrt{A}}{T}$$

A = the drainage area in square miles, and

R = the total flood run-off in inches on the drainage area

The values recommended for R are as follows:

For occasional floods, 3 inches (25 to 75 year frequency).

For rare floods, 6 inches (50 to 200 year frequency).

For maximum flood, 8 inches.

The principal difference between this and the other formulæ is the introduction of the term R , which affords a unit of measure, making it possible to compare floods on different watersheds.

A discussion of the variations in the value of the flood coefficient C_F is also included, and values for it have been determined for a number of New England streams.

This proposed formula is based upon the fundamental conception that peak flows tend to vary directly with the total volume of flood run-off. Exceptions to this tendency are pointed out. In making a comparison of the flood run-off, proper attention must be given to the effect of the base flow, and the effect of storms which are of longer duration than the concentration period.

A comparison of the 1927 flood with the greatest known floods on the principal rivers of New England, and also on some of the smaller streams, has been made. It appears that the 1927 flood had a concentrated storm run-off of between 3 and 6 inches in those areas where the greatest damage was done. Most of the streams of New England have had an occasional flood run-off of between 3 and 4 inches. The only case where the concentrated run-off had exceeded 4 inches during the previous 50 years was the 1923 flood on the Penobscot River, when the figure reached about 5.5 inches, due, however, partly to melting snow. The Miami River, in the great flood of 1913, had a flood run-off of 8 inches. It further appears that a flood run-off of between 3 and 4 inches occurs about once in 50 years in New England. Such floods usually

cause a great deal of inconvenience and considerable damage, but do not generally assume the proportions of a catastrophe. Apparently, most of the river channels and structures are taxed severely by the occasional 3 to 4 inch flood, but can usually be counted upon to come through without great damage. Floods producing a flood run-off of 6 inches and over in New England will inevitably overtax the capacity of river channels and cause great loss. The amount of damage and the loss of life will depend entirely upon the local conditions.

The Committee recommends that floods should be classified in inches of total flood run-off, as a proper basis for the discussion of floods and flood prevention measures. In presenting these formulæ the Committee would emphasize the fact that the flood situation at any point on any stream presents a problem of its own. No general formula can be of universal application. It is only by special study of the conditions at the particular point under consideration that the best results can be obtained. The flood hydrographs of previous floods, when studied in connection with the nature and type of the storms producing these floods, are believed to afford the best information for solving the problem.

Chapter VI — "Flood Characteristics of New England Streams." The geological history of New England, with particular reference to its influence upon the nature and construction of the river systems, is briefly summarized. The remainder of this chapter gives the salient features of the flood characteristics of a number of the larger New England drainage areas upon which flood damage is likely to be most serious.

Chapter VII — "Flood Prevention Measures." These have been discussed with particular reference to New England. This chapter includes numerous examples of what took place during the flood of 1927, by way of illustration of different methods and principles. The Committee has been forced to the conclusion that conditions in New England do not warrant the general application of many of the flood prevention measures used in other parts of the country, excepting, perhaps, in a very few cases where property values are high. This applies to levees, detention reservoirs and channel improvements.

The lakes and storage reservoirs of New England developed for power purposes have been and are of great value in reducing or preventing floods on some of the river systems. The Committee believes that the construction of large water supply and power storage reservoirs constitutes the most effective and practical general method of flood protection in New England. The storms which have produced the great floods have in most cases occurred at the time of the year when

power reservoirs are normally only partly filled, and thus function directly to reduce flood effect. For example, the Davis Bridge Reservoir on the Deerfield River unquestionably prevented a most disastrous flood in 1927. Where it is economically feasible, the upper part of the reservoirs might be reserved for flood prevention. If power storage reservoirs are designed and constructed in part for the purposes of flood protection, it is a question whether the communities benefited should not share a portion of the cost of construction. This raises a question of general policy which, in the judgment of the Committee, should receive serious consideration by the various New England States.

Chapter VIII — "Cost and Economic Phases." To provide fully against the ultimate flood is an impossibility. The flood problem is the common one of economics met with in many other engineering projects. It is an economic impossibility at the present time to prevent highway accidents by the complete elimination of all grade crossings and the construction of single-way roads. Nor can the forces of nature be economically controlled to the extent that inconvenience will not be suffered as a result of heavy storms. Generally speaking, a storage capacity of 3 to 4 inches on the drainage area will eliminate practically all damage due to 3 or 4 inches of flood run-off, and will reduce the damage from 6 and 8 inch floods to moderate proportions. A storage capacity of 4 inches on the drainage area represents approximately 10 million cubic feet per square mile. Its cost in New England will usually be between \$600 and \$1,000 per million cubic feet, making the cost of such flood prevention between \$6,000 and \$10,000 per square mile. This is far beyond the amount that is in most cases warranted for flood prevention alone, even taking into account the frequency of destructive floods.

The above costs would apply to the construction of storage reservoirs solely for the purpose of flood prevention. By combining the use of storage for power and flood prevention it may be possible in some cases to obtain the necessary storage to keep the great floods within bounds, at a substantially lower cost.

Reservoirs even if built solely for power use afford a large measure of flood protection, if operated as they should be with good judgment. Moreover, storage above spillway level, together with intelligent gate operation, will alone greatly cut down flood peaks. This flood remedy is becoming more and more available as storage development progresses, and warrants the encouragement in every way possible of comprehensive power and storage development of our river systems.

Conclusions. — Much of New England is subject to little flood

damage. The great lake regions of Maine and New Hampshire and the coastal regions have little, except for the occasional damage from winds and tides occurring when the coast streams are full. Such other streams as have broad valleys, with small contributing drainage, are reasonably safe from freshets and damage; but the slopes of the White and Green Mountains and the Berkshires have possibilities of great precipitation and rapid run-off, which together constitute a perpetual flood menace.

If the Committee had been given the problem of one river and the effect of the 1927 flood on the different communities and interests in the valley, the answer could be more specific. When, however, the field is all New England, the subject becomes one of only general recommendation.

Some plan of co-operative records and flood warnings should be inaugurated at once to prevent a recurrence of such loss of life as occurred in 1927. Every known means of disseminating information about storms and floods should be used; and, in addition to the Weather Bureau, the records and daily knowledge of the different rivers by the public service companies should be made available through co-operation with them.

The Committee also suggests as relief measures the publication of flood level information in cities and towns along the rivers, statutory restriction of building along river banks which seriously restrict the flood channel, and the extension of state supervision of dams to include maintenance and operation as well as construction. The need of more careful consideration of all structures along and across our New England rivers from the hydraulic point of view as to their effect upon flood heights was emphasized by the 1927 flood.

The Committee does not find that any extensive system of general flood control is warranted, though flood control works may be advisable locally in special cases. The most hopeful general measure toward diminishing flood damage here in New England is to stimulate to the greatest extent possible the development of power storage reservoirs. Vermont has already started this movement, and by the co-operation of Federal and state authorities and public service corporations is developing comprehensive plans for such reservoirs on its rivers.

Legislation to aid in the formation of storage regulating districts (possibly such as have been established in New York State), with provision for proper allocation of cost in accordance with results secured, whether for power or flood protection, appears to be the next logical step.

If the other New England States will co-operate in this movement, it will pave the way for not only the economic development of our river

systems, but also achieve the possible reduction of flood hazard to a point where this will be unimportant. This general program will require a long time to complete, but should be investigated, properly planned, and carried out, step by step, in the future as one of the essential measures toward the prosperity of New England.

Chapter II. — The 1927 Flood in New England

THE STORM OF NOVEMBER 3-4, 1927

The results of a detailed study of the storm of November 3-4, 1927,* show that three factors joined in producing an extraordinary rainfall:

1. A large supply of damp, warm air filling a low-pressure area over the Gulf Stream and South Atlantic region.
2. The piling up of a great and almost immovable mass of cold, high-pressure air in the region lying eastward from the flooded area, including Maine, Nova Scotia and Newfoundland.
3. A southeasterly flow of a considerable body of cold air into northern New England and to the Middle Atlantic coast.

For three days a northward flow of tropical air was funneled into western New England between the stationary air masses on the east and west, forcing great masses of damp air together and up over the obstructing mountains in Vermont and western Massachusetts, as well as over their northern and western bulwarks of cold air, — while at the same time they were being overturned by the cold air above, — resulting in an unusual rainfall in a comparatively short time.

The storm first appeared on the weather map of Wednesday, November 2, with the low center in southern Florida. On Thursday the low center reached Cape Hatteras, and on Friday at 8 A.M. (see Fig. 1) it centered approximately in Vermont. By Saturday it was somewhat north of Quebec. A secondary storm center also caused excessive rainfall in Rhode Island and as far north as Worcester, Mass.

The storm was therefore of the Atlantic coast type, — of tropical origin, but passed farther inland in New England than the usual storm of this class.

* Weber and Brooks, *Journal of New England Waterworks Association*, March, 1928, pp. 91-103.

RAINFALL IN STORM OF NOVEMBER 3-4, 1927*

The greatest recorded precipitation for this storm occurred in Vermont, with a maximum of 9.65 inches at Somerset. It is probable, however, that as much as 11 or 12 inches fell in the higher portions of the Green Mountains. The secondary storm area adjacent to Rhode Island also showed nearly as great a maximum, — that of 9.37 inches at Westerly, R. I.

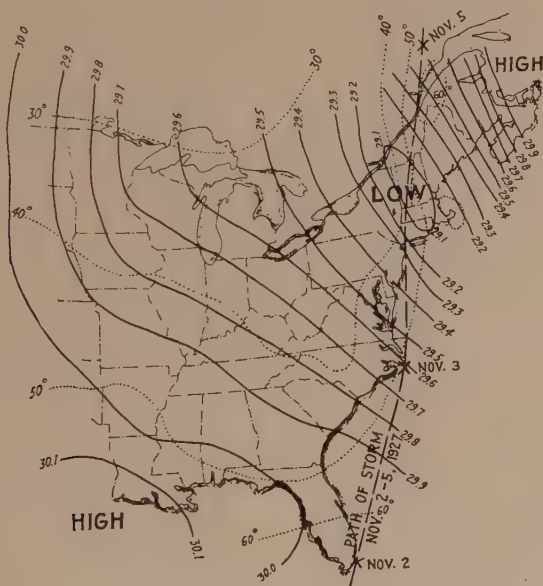


FIG. 1. — DAILY WEATHER MAP FOR NOVEMBER 4, 1927

The isohyets for this storm are shown on Fig. 2. For the State of Vermont (about 9,564 square miles in area), where the greatest flood damage occurred, the average rainfall was about 6.0 inches. Data regarding rainfall-area relations are given in Table 8.

The rainfall in the month of October, 1927, was practically double the normal amount in the flood area, a factor of great importance in producing the high rates of run-off in the November flood, as the ground was already saturated and streams and ponds generally full. This is shown by Table 1.

* For more complete data see "Rainfall in New England during the Storm of November 3 and 4, 1927," X. H. Goodnough, Journal of New England Waterworks Association, June, 1928.

TABLE 1. — RAINFALL, OCTOBER, 1927

LOCALITY	Number of Stations	Total Rainfall, October, 1927	Normal October Rainfall
Vermont	3	5.99	3.35
Western Massachusetts	5	7.13	3.58
East-Central Massachusetts	4	5.73	3.28

While rain fell practically continuously in the storm of November, 1927, for from 28 to 40 hours, the bulk of the rainfall occurred generally in 10 to 18 hours, and some high rates of rainfall occurred in many localities. Some of the more striking results are given in Table 2.

TABLE 2. — RAINFALL RATES, STORM OF NOVEMBER 2-5, 1927

STATION	Time (Hours)	Rate per Hour (Inches)	MAXIMUM	
			Time (Hours)	Inches (Per Hour)
Munroe Bridge, Mass.	16	.28	2	.39
Northfield, Vt.	9	.44	1	.62
Searsburg, Vt.	22	.29	1	.59
Somerset, Vt.	12	.44	2	.66
Worcester, Mass.	5	.90	1	1.42

Thus, in Vermont the rainfall, which averaged nearly .50 inch per hour, occurred for many consecutive hours, and in the Blackstone River valley in Massachusetts and Rhode Island it was even more excessive.

In Table 3 are given data of rainfall by days for the November, 1927, storm for the five stations in Vermont and in Rhode Island where rainfall was highest, in order of magnitude for the storm period.

TABLE 3.—RAINFALL IN STORM OF NOVEMBER 2-5, 1927

STATION	RAINFALL IN INCHES				Total, Nov. 2-4	Total, Nov. 2-5
	Nov. 2	Nov. 3	Nov. 4	Nov. 5		
<i>Vermont</i>						
Somerset	.00	.68	8.77	.20	9.45	9.65
Northfield	—	6.17	2.46	.03	8.63	8.66
Mollys Falls	—	—	—	—	—	9.14
Chittenden	1.65	6.60	.35	.00	8.60	8.60
Rutland	—	1.90	6.12	.45	8.47	8.47
<i>Rhode Island</i>						
Westerly	—	—*	9.37	—	9.37	9.37
Westerly (Water Works)	—	—*	9.12	—	9.12	9.12
Hopkins Mills	—	.07	7.85	.02	7.92	7.94
Kent	—	.11	7.51	.09	7.62	7.71
Rock Hill	—	.02	6.86	.02	6.88	6.90

* Included in following day's total.

Rainfall-Altitude Relation

The highest elevation of any New England rainfall station in 1927 was at Somerset, Vt., El. 2,096. To study the effect of altitude on storm rainfall, Fig. 2*a* was prepared, where total storm rainfall for November 2-5, 1927, is plotted against elevation for Vermont stations. While some points are discordant, a reasonably good extension curve to about El. 3,000 is shown in the form of —

$$P = 6.6 + (E - 5) \times 0.18$$

where
and

P = storm rainfall in inches

E = elevation in hundreds of feet.

This indicates rainfalls at the higher elevations, as follows:

ELEVATION	Rainfall
1,000	7.5
1,500	8.4
2,000	9.3
2,500	10.2
3,000	11.1

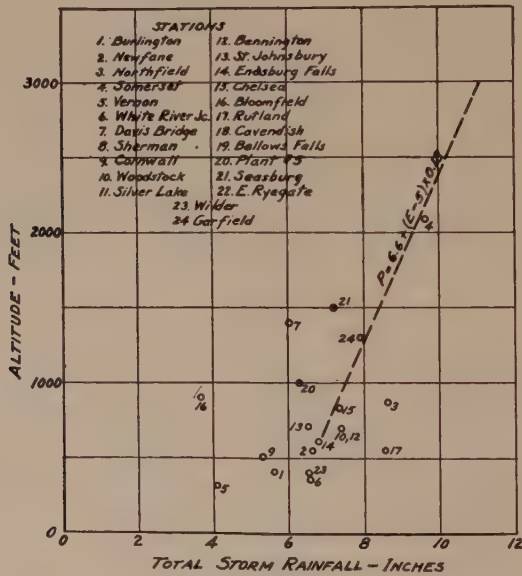


FIG. 2a. — RAINFALL-ALTITUDE RELATIONS IN VERMONT

Thus, for the Deerfield River drainage area of 30 square miles, above Somerset Reservoir, which has an average elevation of 2,400 feet, the storm rainfall can be estimated at about 10.0 inches. The estimated storm run-off at this point was about 6 inches — which is fairly consistent.

This storm rainfall equation would apply only to Vermont. What happened in New Hampshire, in the White Mountains, is only a matter of conjecture, as no high level records were obtained. The need for

higher altitude rainfall stations in New England as a basis for intelligent study of this matter is obvious.

There were certain rainfall records not included in the paper on rainfall in New England during the storm of November 3-4, 1927,* that are given below. Some of these records are not consistent with the rainfall contours as shown on Fig. 2. In some cases the methods of measurement were not standard, and the records may not be accurate, but they suggest the possibility that the rainfall may have been "spotty," and also, perhaps, much heavier in some localities than is indicated by the contours in Fig. 2.

REPORTED RAINFALL RECORDS, NOVEMBER 2-5, 1927, NOT PREVIOUSLY PUBLISHED

PLACE	Depth in Inches	Remarks
<i>Vermont</i>		
Castleton	11.00	Measured in a lard tub
Milton	8.00	
Swanton	4.10	
Manchester	7.50	Approximate
Hyde Park	8.00	
Marshfield	8.25	
Derby Line	10.5	Measured in an 8-inch glass cylinder with readings taken at 3-hour intervals.
Quarry near Rochester	8.00 (or more)	12-quart pail overflowed
<i>New Hampshire</i>		
Dover	3.10	
Lyme	6.50	
Rollinsford	8.00	
Stratford	4.50	
Milton	6.00	
Portsmouth	1.98	
Pembroke	4.50	
Lost River near Mt. Moosilauke	15.00	El. 1,600 feet. Measured in a barrel.

* "Rainfall in New England during the Storm of November 3 and 4, 1927," X. H. Goodnough, Journal of New England Waterworks Association, June, 1928.

RUN-OFF FROM STORM OF NOVEMBER 2-5, 1927

At the time of the flood the United States Geological Survey, in co-operation with the States of Maine, New Hampshire and Massachusetts, was operating about 60 stream-flow measurement stations in New England at which records of peak stages were obtained. While the flood was in progress a number of discharge measurements were made at stations in Massachusetts. As soon as it was possible to travel over the roads in the flooded area, field parties were sent to the sections affected by the storm to determine maximum stages and discharges at gaging stations, dams and other points on the principal streams. These determinations and the method used in securing them, together with a complete report of the flood and its causes, have been published by the United States Geological Survey.*

The methods used in securing these data can be described briefly as follows:

Extension of Rating Curves for Gaging Stations. — At regular gaging stations, the rating curves had previously been determined to medium or fairly high stages by current meter measurements. Where conditions warranted, these rating curves were extended to peak stages, and the maximum discharge determined from the curve.

Computation of Flow Over Dams. — Where conditions at a dam permitted, the length of the spillway and the area of the approach channel were obtained by a survey, and the area and head for all gates and other openings ascertained. With this information, the peak discharge was computed by weir and orifice formulæ, with allowance for velocity of approach in the case of the former.

Contracted Opening Method. — At various points where a river was forced to pass through a contracted section of the river channel, usually between bridge abutments, the maximum discharge was determined by a survey of the cross section at its most contracted point, the cross-sectional area in the channel of approach, and the elevation of the water surface immediately above and below. With this information, the flow was computed by multiplying the area of the most contracted section by the velocity through it. The latter was computed from the velocity head shown by the "drop-off" plus the head due to velocity of approach less the friction head.

Slope Area Method. — Where conditions were not favorable for the determination of discharge by a more accurate method, a survey

* H. B. Kinnison, District Engineer, Water Supply Paper 636, Section C.

was made of an unobstructed and uniform section of the river channel, varying in length from a few hundred feet to half a mile long, in which was determined the slope of the river at the time of the peak stage, several cross sections of the river to peak stage, and the roughness of the stream bed. From this information the mean velocity was computed by means of the Chezy-Kutter formula $V = C\sqrt{RS}$.

The results of these determinations, with other essential information, are shown in Table 4. Flood hydrographs are shown in Figs. 3, 4, 13 and 14, and flood profiles in Table 5.

TABLE 4. — MAXIMUM DISCHARGE AND TOTAL RUN-OFF

No. on Map	RIVER AND POINT OF MEASUREMENT *	Period of Record	MAXIMUM DISCHARGE PREVIOUSLY RECORDED	
			Date	Discharge (Second- Feet)
Penobscot River Basin				
1	Penobscot, West Enfield, Me.	1901-1927	May 1, 1923	153,000
2	East Branch of Penobscot, Grindstone, Me.	1902-1927	Apr. 30, 1923	35,100
3	Mattawamkeag, Mattawamkeag, Me.	1902-1927	May 1, 1923	43,900
4	Piscataquis, Lows Bridge, near Foxcroft, Me.	1902-1927	Sept. 29, 1909	21,700
5	Piscataquis, Medford, Me.	1924-1927	Oct. 21, 1927	24,600
6	Pleasant, Milo, Me.	1920-1927	Apr. 30, 1923	24,400
Kennebec River Basin				
7	Kennebec, Waterville, Me.	1892-1927	— —	—
8	Dead, The Forks, Me.	{ 1901-1907 1910-1927 }	Apr. 30, 1923	23,800
9	Carrabasset, near North Anson, Me.	1925-1927	May 3, 1926	8,590
Androscoggin River Basin				
10	Androscoggin, upper plant of Brown Company, Berlin, N. H.	1913-1922	— —	—
11	Androscoggin, dams of Rumford Falls Power Company, Rumford, Me.	1892-1927	— —	—
12	Androscoggin, dam of Central Maine Power Company, Lewiston, Me.	— —	1896	65,000
13	Peabody, above Nineteenmile Brook, near Glen House, N. H.	— —	— —	—
14	Peabody, below Barnes Brook, near Gorham, N. H.	— —	— —	—
Saco River Basin				
15	Saco, Cornish, Me.	1916-1927	May 2, 1923	23,000
16	Ellis, above Wildcat Brook, Jackson, N. H.	— —	— —	—
17	Wildcat Brook, 2.1 miles above Jackson, N. H.	— —	— —	—
18	Ossipee, Cornish, Me.	1916-1927	Apr. 30, 1923	6,740
Merrimack River Basin				
19	Pemigewasset, below Bakers River, at Plymouth, N. H.	1886-1927	Apr. 29, 1923	28,000
20	Pemigewasset, dam at Bristol, N. H.	— —	— —	—
21	Pemigewasset, dam at Franklin Falls, N. H.	— —	— —	—
22	Merrimack, Franklin Junction, N. H.	1903-1927	Apr. 30, 1923	43,700
23	Merrimack, dam at Sewalls Falls, N. H.	— —	— —	—

* Reduced by amount of regulated flow, or leakage.

¹ Record furnished by Hollingsworth & Whitney Company, Waterville, Me.

² Record furnished by P. L. Bean, Union Water Power Company, Lewiston, Me.

DURING THE NEW ENGLAND FLOOD OF NOVEMBER, 1927

DRAINAGE AREA (SQUARE MILES)		FLOOD OF NOVEMBER, 1927				
Total	Effective Area for Flood of No- vember, 1927	MAXIMUM DISCHARGE			Total Run-off Nov. 3-10 (Millions of Cubic Feet)	Method of Determination
		Time of Flood Crest	Maximum (Second- Feet)	Per Square Mile (Second- Feet)		
6,600	4,690	Nov. 6	60,800	12.5 ¹	23,682	Rating curve.
1,070	1,070	Nov. 5	21,300	19.9	5,710	Rating curve.
1,500	1,500	Nov. 6	16,900	11.3	7,628	Rating curve.
286	286	Nov. 5	6,740	23.6	1,447	Rating curve.
1,170	1,170	Nov. 5	18,200	15.6	5,572	Rating curve.
325	325	Nov. 5	7,260	22.4	2,300	Rating curve.
4,270	3,030	Nov. 5	76,800	26.3 ¹	22,366	Flow over dam, through wheels and gates. ²
878	830	Nov. 5	14,700	17.7	5,790	Rating curve.
351	351	Nov. 4, 10 P.M.	18,600	53	2,407	Rating curve.
1,380	538	Nov. 4, 8-10 P.M.	12,000	22.3	4,186	Weir formula. ³
2,090	1,248	Nov. 5	46,700	37.4	9,707	— — — ⁴
2,856	2,014	Nov. 5, 7 P.M.	60,000	29.8	14,604	Flow over dam, through wheels and gates. ³
17.4	17.4	— —	7,330	421	—	Slope-area; two determi- nations, $n=0.06$ and 0.07.
40	40	— —	9,920	248	—	Slope-area, $n=0.07$.
1,300	1,300	Nov. 7	10,800	8.3	4,961	Rating curve.
28	28	— —	14,800	528	—	Slope-area, $n=0.035$.
15	15	— —	4,080	272	—	Contracted opening.
455	455	Nov. 6	2,320	5.1	1,133	Rating curve.
615	599	Nov. 4, 5 P.M.	60,000	100	7,156 ⁵	Flow over Bristol dam re- duced by estimated in- flow.
760	686	— —	62,300	91	—	Dam; $C=3.83$.
956	791	— —	59,700	75	—	Dam; $C=3.8$.
1,460	935	Nov. 5, 2 A.M.	67,000	71 ¹	10,780 ⁵	Rating curve.
2,280	1,755	Nov. 5, 10 A.M.	58,000	32.7 ¹	—	Dam; $C=2.60$. ⁶

⁴ Record furnished by Charles A. Mixer, Rumford Falls Power Company, Rumford, Me.⁵ Partly estimated.⁶ Record furnished by H. M. Turner, consulting engineer, Boston, Mass.

TABLE 4. — MAXIMUM DISCHARGE AND TOTAL RUN-OFF DURING

No. on Map	RIVER AND POINT OF MEASUREMENT	Period of Record	MAXIMUM DISCHARGE PREVIOUSLY RECORDED	
			Date	Discharge (Second- Feet)
Merrimack River Basin — Continued				
24	Merrimack, dam at Garvins Falls, N. H.	— —	— —	—
25	Merrimack, Manchester, N. H.	1924-1927	— —	—
26	Merrimack, Lowell, Mass.	{ 1848-1861 1866-1916 }	Apr. 23, 1852	83,000
27	Merrimack, dam of Essex Company, Lawrence, Mass.		Mar. 3, 1896	86,900
28	Mad, 4 miles above Campton Village, N. H.	— —	— —	—
29	Mad, dam of electric plant, Campton Village, N. H.	— —	— —	—
30	Beebe, 1 mile east of Beebe River, N. H.	— —	— —	—
31	Bakers, 2 miles north of Wentworth, N. H.	— —	— —	—
32	South Branch of Bakers, near mouth, West Rum- ney, N. H.	— —	— —	—
33	Smith, 3 miles southwest of Bristol, N. H.	1918-1927	Mar. 29-30, 1925	2,260
34	Nubanusit Brook, 1½ miles above Peterboro, N. H.	1920-1927	Mar. 10, 1921	1,050
35	North Branch of Contoocook, North Branch Village, 4 miles northwest of Antrim, N. H.	1924-1927	Apr. 26, 1926	1,600
36	Suncook, North Chichester, N. H.	1918-1927	Apr. 7, 1923	4,300
37	Souhegan, Merrimack, N. H.	1909-1927	Apr. 8, 1924	10,400
Providence River Basin				
38	Blackstone, Worcester, Mass.	1923-1927	Apr. 7, 1924	740
Pawtuxet River Basin				
39	Pawtuxet, Scituate Dam at Kent, R. I.	— —	— —	—
40	Ponaganset, Barden Reservoir, Ponaganset	— —	— —	—
Thames River Basin				
41	Quinebaug, Jewett City, Conn.	1918-1927	Mar., 1920	12,000
Connecticut River Basin				
42	Connecticut, Waterford, Vt.	1927	— —	—
43	Connecticut, South Newbury, Vt.	1918-1927	May 1, 1923	56,700
44	Connecticut, dam at Wilder, Vt.	— —	— —	—
45	Connecticut, White River Junction, Vt.	1911-1927	Mar. 27, 1913	113,000
46	Connecticut, Vernon, Vt.	— —	— —	—

¹ Reduced by amount of regulated flow, or leakage.² Record furnished by H. M. Turner, consulting engineer, Boston, Mass.³ Record furnished by Amoskeag Manufacturing Company, Manchester, N. H.⁴ Record furnished by Proprietors of Locks and Canals on Merrimack River, Lowell, Mass.

THE NEW ENGLAND FLOOD OF NOVEMBER, 1927 — *Continued*

DRAINAGE AREA (SQUARE MILES)		FLOOD OF NOVEMBER, 1927				
Total	Effective Area for Flood of Nov- ember, 1927	MAXIMUM DISCHARGE			Total Run-off Nov. 3-10 (Millions of Cubic Feet)	Method of Determination
		Time of Flood Crest	Maximum (Second- Feet)	Per Square Mile (Second- Feet)		
2,340	1,815	Nov. 5, 4 P.M.	58,500	31.9 ¹	—	Dam; $C=3.8$ and 3.6 . ²
2,840	2,320	Nov. 5, 9 P.M.	60,300	26.0 ¹	—	Rating curve. ³
4,215	3,570	Nov. 6, 4 A.M.	73,000	20.3 ¹	—	— — ⁴
4,663	3,930	Nov. 6, 4 A.M.	70,360	17.6 ¹	—	Rating curve for dam. ⁵
47	47	— —	10,200	217	—	Slope-area; $n=0.055$.
59	59	— —	12,000	203	—	Dam; $C=3.33$.
31	31	— —	5,800	187	—	Contracted opening.
52	52	— —	15,000	288	—	Slope-area; $n=0.04$ and 0.03.
42	42	— —	9,800	233	—	Contracted opening.
78.5	78.5	— —	5,800	74	595 ⁶	Rating curve and slope- area.
54.3	54.3	Nov. 4, 10 P.M.	1,010	19	276	Rating curve.
59.5	59.5	Nov. 5, 7 A.M.	2,100	35	430 ⁶	Rating curve.
157	157	— —	1,860	11.8	—	Rating curve.
168	168	Nov. 5, 1 A.M.	6,650	40	952	Rating curve.
31.5	31.5	Nov. 4, 6-7 P.M.	790	25	132	Rating curve.
92.8	83.8	Nov. 4, 8-10 A.M.	10,900	130	—	Storage in reservoirs. ⁷
33	32	Nov. 4, 7 A.M.	4,320	135	—	Dam; $C=3.5$. ⁷
712	712	Nov. 5, 4 A.M.	12,500	18	3,434	Rating curve.
1,600	1,520	Nov. 6, 8 A.M.	31,100	20.5	—	Rating curve.
2,830	2,750	Nov. 4, 12 P.M.	78,000	28	23,004 ⁶	Flow over Wilder Dam reduced by estimated inflow.
3,410	3,330	Nov. 5, 12 M.	83,700	25	—	Dam; $C=2.8$.
4,120	4,040	Nov. 4, 7 A.M.	148,000	37	38,552 ⁶	Rating curve.
6,300	6,220	Nov. 5, 1 A.M.	155,000	24.9	—	Rating curve.

⁵ Record furnished by Essex Company, Lawrence, Mass.⁶ Partly estimated.⁷ Record furnished by W. W. Peabody, Water Supply Board, Providence, R. I.

TABLE 4. — MAXIMUM DISCHARGE AND TOTAL RUN-OFF DURING

No. on Map	RIVER AND POINT OF MEASUREMENT	Period of Record	MAXIMUM DISCHARGE PREVIOUSLY RECORDED	
			Date	Discharge (Second- Feet)
Connecticut River Basin — Continued				
47	Connecticut, Turners Falls, Mass.	— —	— —	—
48	Connecticut, Sunderland, Mass.	1904-1927	Mar. 28, 1913	135,000
49	Connecticut, Holyoke, Mass.	— —	— —	—
50	Mohawk, 6.15 miles above Colebrook, N. H. . .	— —	— —	—
51	Mohawk, bridge 5.75 miles above Colebrook, N. H.	— —	— —	—
52	Mohawk, 4.9 miles above Colebrook, N. H. . .	— —	— —	—
53	Israel, dam at Lancaster, N. H.	— —	— —	—
54	Johns, dam at Whitefield, N. H.	— —	— —	—
55	Passumpsic, Pierce's mills, near St. Johnsbury, Vt.	— —	— —	—
56	Ammonoosuc, Bethlehem Electric Company's dam, near Bethlehem, N. H.	— —	— —	—
57	Ammonoosuc, upper dam of Littleton Light and Power Company, 2 miles above Littleton, N. H.	— —	— —	—
58	Waits, Bradford Electric Light Company dam, Bradford, Vt.	— —	— —	—
59	White, 2 miles below West Hartford, Vt. . .	— —	— —	—
60	Mascoma, Mascoma, N. H.	1923-1927	Mar. 30, 1925	3,700
61	Ottawaquechee, dam at Taftsville, Vt. . . .	— —	— —	—
62	Ottawaquechee, dam at Deweys Mills, Vt. . .	— —	— —	—
63	West, 1¼ miles northeast of Newfane, Vt. . .	1919-1927	Apr. 12, 1922	12,200
64	West, dam at West Dummerston, Vt.	— —	— —	—
65	Ashuelot, 1 mile below Gilsum, N. H. . . .	1922-1927	Apr. 25, 1926	1,350
66	Ashuelot, dam 1½ miles above Hinsdale, N. H.	— —	— —	—
67	South Branch of Ashuelot, dam at Troy, N. H. .	— —	— —	—
68	South Branch of Ashuelot, at Webb near Marlboro, N. H.	1920-1927	Feb. 12, 1925	1,680
69	Millers, Erving, Mass.	1914-1927	Mar. 28, 1920	6,020
70	Sip Pond Brook, 3 miles northwest of Winchendon, Mass.	1916-1927	May 23, 1919	339
71	Priest Brook, 3½ miles west of Winchendon, Mass.	1916-1927	Mar. 28, 1919	700
72	East Branch of Tully River, 3½ miles north of Athol, Mass.	1916-1927	Mar. 29, 1920	1,000
73	Moss Brook, Wendell Depot, Mass.	1916-1927	Mar. 28, 1919	190
74	Deerfield, Somerset Reservoir, Vt.	— —	— —	—
75	Deerfield, Davis Bridge Reservoir, Vt. . . .	— —	— —	—
76	Deerfield, Charlemont, Mass.	1913-1927	July 8, 1915	50,600
77	Cold, 2¾ miles south of Hoosac Tunnel, Mass. .	— —	— —	—
78	Cold, near mouth, 2½ miles northwest of Charle- mont, Mass.	— —	— —	—
79	Ware, Gibbs Crossing, 3 miles below Ware, Mass.	1912-1927	Apr. 8, 1924	2,950
80	Swift, West Ware, Mass.	1910-1927	Apr. 7, 1923	2,390

¹ Record furnished by H. A. Moody, Turners Falls Power & Electric Company.² Partly estimated.

THE NEW ENGLAND FLOOD OF NOVEMBER, 1927 — *Continued*

DRAINAGE AREA (SQUARE MILES)		FLOOD OF NOVEMBER, 1927					
Total	Effective Area for Flood of No- vember, 1927	MAXIMUM DISCHARGE			Total Run-off Nov. 3-10 (Millions of Cubic Feet)	Method of Determination	
		Time of Flood Crest	Maximum (Second- Feet)	Per Square Mile (Second- Feet)			
7,250	7,170	- -	171,000	23.8	-	Dam; $C=3.9$. ¹	
8,000	7,740	Nov. 5, 4 P.M.	165,000	21.3	59,132	Rating curve.	
8,390	8,120	Nov. 5, 2.30-4.30 P.M.	169,000	20.8	-	Flow at Sunderland plus estimated inflow.	
26	26	- -	4,340	167	-	Slope-area; $n=0.03$.	
30.5	30.5	- -	5,110	167	-	Contracted opening.	
32	32	- -	5,310	166	-	Slope-area; $n=0.03$.	
124	124	- -	8,840	71	-	Dam; $C=3.6$.	
35	35	- -	1,080	31	-	Dam; $C=3.5$.	
237	237	- -	33,000	139	-	Dam; $C=3.9$.	
97	97	- -	17,900	185	-	Dam; $C=2.7$ and 3.85 .	
124	124	- -	16,600	134	-	Dam; $C=3.74$.	
162	162	- -	10,500	65	-	Dam; $C=3.50$.	
695	695	Nov. 4, 3 A.M.	140,000	202	798	Slope-area; $n=0.032$.	
148	148	Nov. 5, 4 P.M.	3,230	21.8	-	Rating curve.	
192	192	- -	25,400	132	-	Dam; $C=4.0$.	
208	208	- -	27,200	131	-	Dam; $C=3.8$.	
310	310	Nov. 3, 12 P.M.	45,000	145	-	Rating curve.	
410	410	- -	49,000	120	-	Dam; $C=3.33$.	
68.5	68.5	Nov. 4, 12 M.	2,760	40	507	Rating curve.	
431	431	Nov. 4, 11.30 A.M.	6,510	15.3	-	Dam; $C=3.8$ and 3.3 .	
8	8	- -	913	114	-	Dam; $C=3.7$.	
36.6	36.6	Nov. 4, 8.15 A.M.	3,560	97	333	Rating curve.	
372	372	Nov. 4, 10.30 A.M.	6,350	17	2,082	Rating curve.	
18.8	18.8	Nov. 4, 7 P.M.	340	18	97	Rating curve.	
18.8	18.8	Nov. 5	1,000	53	195 ¹	Rating curve.	
50.2	50.2	Nov. 4, 5 P.M.	1,610	32	360 ²	Rating curve.	
12.2	12.2	Nov. 4, 4.45 P.M.	775	64	144 ²	Rating curve.	
30	30	- -	-	165 ³	-	Storage in reservoir. ¹	
184	154	- -	-	200 ⁴	-	Storage in reservoir. ¹	
362	180	Nov. 3, 9.30 P.M.	36,000	200	3,163	Rating curve.	
22.4	22.4	- -	6,870	307	-	Slope-area; $n=0.065$.	
32.2	32.2	- -	7,760	241	-	Slope-area; $n=0.08$.	
201	201	Nov. 4, 9 P.M.	2,830	14.1	866	Rating curve.	
186	186	Nov. 6, 3 A.M.	2,230	12.0	839	Rating curve.	

¹ Based on increase in storage from 4 P.M., November 3 to 5 A.M., November 4.² Based on increase in storage from 2 to 5 A.M., November 4.

TABLE 4. — MAXIMUM DISCHARGE AND TOTAL RUN-OFF DURING

No. on Map	RIVER AND POINT OF MEASUREMENT	Period of Record	MAXIMUM DISCHARGE PREVIOUSLY RECORDED	
			Date	Discharge (Second- Feet)
Connecticut River Basin—Concluded				
81	Quaboag, West Brimfield, Mass.	1909-1927	Mar. 17, 1920	1,980
82	Westfield, dam at Cummington, Mass.	— —	— —	—
83	Westfield, dam at West Chesterfield, Mass.	— —	— —	—
84	Westfield, dam at Huntington, Mass.	— —	— —	—
85	Westfield, dam at Russell, Mass.	— —	— —	—
86	Westfield, Trap Rock Crossing, near Westfield, Mass.	1914-1927	Apr. 7, 1924	32,500
87	Westfield, dam at Mitteneague, Mass.	— —	— —	—
88	Stevens Brook, dam at West Chesterfield, Mass.	— —	— —	—
89	Little, upper and lower dams at South Worthington, Mass.	— —	— —	—
90	Middle Branch of Westfield, Goss Heights, Mass.	1910-1927	July 8, 1915	4,500
91	West Branch of Westfield, dam at Huntington, Mass.	— —	— —	—
92	Walker Brook, dam at Chester, Mass.	— —	— —	—
93	Westfield Little, diversion dam of Springfield waterworks, 3 miles west of Westfield, Mass.	1905-1922	Mar. 13, 1920	1,940
94	Westfield Little, Crane Paper Company's dam, Westfield, Mass.	— —	— —	—
95	Farmington, 1 mile south of New Boston, Mass.	1913-1927	Apr. 7, 1924	3,450
96	Farmington, New Hartford, Conn.	— —	— —	—
97	Farmington, dam at Rainbow, Conn.	— —	— —	—
98	East Branch of Farmington, dam at Barkhamstead, Conn.	— —	— —	—
99	Nepaug, above Nepaug Reservoir, near Collinsville, Conn.	— —	— —	—
100	Phelps Brook, above Clear Brook, near Collinsville, Conn.	— —	— —	—
101	Clear Brook, $\frac{1}{4}$ mile above mouth, near Collinsville.	— —	— —	—
Housatonic River Basin				
102	Housatonic, dam at Dalton, Mass.	— —	— —	—
103	Housatonic, dam $1\frac{1}{2}$ miles west of Dalton, Mass.	— —	— —	—
104	Housatonic, dam at Lee, Mass.	— —	— —	—
105	Housatonic, dam at South Lee, Mass.	— —	— —	—
106	Housatonic, Falls Village, Conn.	1912-1927	Mar. 29, 1914	8,830
Hudson River Basin				
107	Batten Kill, 1 mile southwest of Battenville, N. Y.	1922-1927	Feb. 12, 1925	7,350
108	Hoosic, $\frac{1}{4}$ mile above North Branch, North Adams, Mass.	— —	— —	—

¹ Partly estimated.² Record furnished by Springfield Waterworks, Springfield, Mass.

THE NEW ENGLAND FLOOD OF NOVEMBER, 1927 — *Continued*

DRAINAGE AREA (SQUARE MILES)		FLOOD OF NOVEMBER, 1927				
Total	Effective Area for Flood of No- vember, 1927	MAXIMUM DISCHARGE			Total Run-off Nov. 3-10 (Millions of Cubic Feet)	Method of Determination
		Time of Flood Crest	Maximum (Second- Feet)	Per Square Mile (Second- Feet)		
150	150	Nov. 4, 8.30 A.M.	1,180	7.9	471	Rating curve.
53	53	— —	6,020	114	—	Dam; $C=3.3$.
99	99	— —	8,720	89	—	Dam; $C=2.8$.
224	224	— —	23,600	105	—	Dam; $C=3.66$.
322	322	— —	30,100	93	—	Dam; $C=3.25$.
496	496	Nov. 4, 8 A.M.	42,500	86	5,037	Rating curve.
512	512	— —	38,000	74	—	Dam; $C=3.8$.
12	12	— —	1,450	121	—	Dam; $C=3.7$.
10.4	10.4	— —	1,230	118	—	Dam; $C=3.8$ for each dam.
53	53	Nov. 3, 9 P.M.	5,860	111	539	Rating curve.
95	95	— —	16,000	168	—	Dam; $C=3.5$.
18	18	— —	2,080	116	—	Dam; $C=3.6$.
48.5	48.5	— —	3,940	81	—	— — ²
81	81	— —	8,500	105	—	Dam; $C=3.6$.
92.7	75.1	Nov. 3, 9.30 P.M.	7,900	105	1,145	Rating curve.
231	213	— —	24,750	116	—	— — ²
581	563	Nov. 5, 4-5 A.M.	22,300	40	—	Dam; $C=3.66$. ⁴
61.2	61.2	— —	8,680	142	—	— — ²
23.9	23.9	— —	1,783	74.6	—	— — ²
2.9	2.9	— —	199	69.5	—	— — ²
1.05	1.05	— —	46	43.8	—	— — ²
54.7	50.4	— —	5,460	108	—	Dam; $C=3.8$.
57	53	— —	4,830	91	—	Dam; $C=3.0$.
187	161	— —	5,980	37	—	Dam; $C=3.7$.
244	218	— —	7,830	36	—	Dam; $C=3.3$.
644	618	Nov. 5, 6 P.M.	11,700	18.9	4,901	Rating curve.
397	397	— —	20,000	50.7	—	Rating curve.
74	74	— —	7,770	105	—	Dam; $C=3.45$.

² Record furnished by C. M. Saville, Board of Water Commissioners, Hartford, Conn.⁴ Record furnished by P. W. Fairbanks, hydraulic engineer, New Britain, Conn.

TABLE 4. — MAXIMUM DISCHARGE AND TOTAL RUN-OFF DURING

No. on Map	RIVER AND POINT OF MEASUREMENT	Period of Record	MAXIMUM DISCHARGE PREVIOUSLY RECORDED	
			Date	Discharge (Second- Feet)
<i>Hudson River Basin — Concluded</i>				
109	Hoosic, Greylock Mills, dams 2 miles west of North Adams, Mass.	— —	— —	—
110	Hoosic, Greylock Mills dam at North Pownal, Vt.	— —	— —	—
111	Hoosic, 1½ miles southeast of Eagle, Bridge, N. Y.	1910-1927	July 9, 1915	16,700
112	North Branch of Hoosic, Hoosic Mills dam, North Adams, Mass.	— —	— —	—
113	Green, Boyd's dam, Williamstown, Mass.	— —	— —	—
114	Poesten Kill, 4½ miles above mouth, 3 miles east of Troy, N. Y.	1923-1927	Feb. 12, 1925	3,280
<i>Lake Champlain Basin</i>				
115	Otter Creek, Middlebury, Vt.	{ 1903-1907 1910-1920 }	Mar. 30, 1913	10,000
116	Otter Creek dam at Huntington Falls, above Weybridge, Vt.	— —	— —	—
117	East Creek, Patch Dam, 2 miles above Rutland, Vt.	— —	— —	—
118	Tenney Brook, Dunklee Pond Dam, near Rutland, Vt.	— —	— —	—
119	Winooski, above Dog River, Montpelier, Vt.	1909-1923	Apr. 7, 1912	20,200
120	Winooski, above Dog River, Montpelier, Vt.	— —	— —	—
121	Winooski, 4½ miles above dam at Essex Junction, Vt.	— —	— —	—
122	Winooski, Burlington Light and Power Company's dam, Essex Junction, Vt.	— —	— —	—
123	Mollys Brook, dam of Montpelier & Barre Light and Power Company, Mollys Falls, Vt.	— —	— —	—
124	Jail Branch, East Barre, Vt.	1920-1923	Apr. 10, 1922	1,350
125	North Branch of Winooski, dam at Wrightsville, Vt.	— —	— —	—
126	Dog, ¾ mile above Union Brook, Northfield, Vt.	1909-1920	Mar. 25, 1913	3,400
127	Dog, dam of Cross Brothers plant, Northfield, Vt.	— —	— —	—
128	Mad, dam, No. 8 of Peoples Electric Company, near Middlesex, Vt.	— —	— —	—
129	Lamoille, dam of Morrisville Electric Light and Power Company, Cadys Falls, Vt.	— —	— —	—
130	Lamoille, dam at Public Electric Light Company, Fairfax Falls, Vt.	— —	— —	—
131	Missisquoi, 2 miles above Trout River, 3 miles below Richford, Vt.	1909-1923	Apr. 7, 1923	16,000
132	Missisquoi, dam at Sheldon Springs, Vt.	— —	— —	—
<i>St. Francis River Basin</i>				
133	Clyde, dam of Newport Electric Light Company, West Derby, Vt.	1909-1924	March, 1913	4,500
134	Tomifobia, Butterfields Company's dam at Derby Line, Vt.	— —	— —	—

¹ Record furnished by H. M. Turner, consulting engineer, Boston, Mass.

THE NEW ENGLAND FLOOD OF NOVEMBER, 1927 — *Concluded.*

DRAINAGE AREA (SQUARE MILES)		FLOOD OF NOVEMBER, 1927				
Total	Effective Area for Flood of November, 1927	MAXIMUM DISCHARGE			Total Run-off Nov. 3-10 (Millions of Cubic Feet)	Method of Determination
		Time of Flood Crest	Maximum (Second- Feet)	Per Square Mile (Second- Feet)		
124	124	- -	12,400	100	-	Two dams; $C=3.33$ for each.
223	232	- -	14,600	66	-	Dam; $C=3.3$.
512	512	- -	29,700	58	-	Rating curve.
40	40	- -	12,200	305	-	Dam; $C=3.3$.
42.3	42.3	- -	3,870	92	-	Dam; $C=3.5$.
88	88	- -	7,150	81	-	Rating curve.
615	590	- -	13,600	23.1	-	Rating curve.
739	714	- -	18,800	26.3	-	Dam; $C=2.64$.
51	26.1	- -	4,750	182	-	Dam; $C=3.2$. ¹
5.2	5.2	- -	900	173	-	Dam; $C=3.37$. ¹
420	397	Nov. 3, 12 P.M.	57,000	144	-	Rating curve.
420	397	- -	60,600	153	-	Slope-area; $n=0.04$.
1,034	1,010	Nov. 4, 2 P.M.	110,000	109	-	Slope-area; $n=0.03$.
1,044	1,020	Nov. 4, 2 P.M.	116,000	114	-	Dam; $C=4.00$.
24	24	Nov. 5, 7 A.M.	581	14.2	-	Dam.
38	38	- -	11,500	303	-	Rating curve.
67	67	Nov. 3, 11 P.M.	17,200	257	-	Dam; $C=3.33$.
52	52	Nov. 3, 6.30 P.M.	8,000	154	-	Rating curve.
60.5	60.5	- -	9,160	151	-	Dam; $C=3.9$.
143	143	Nov. 3, 12 P.M.	23,000	161	-	Dam; $C=3.00$.
280	280	- -	36,600	131	-	Dam; $C=4.0$.
559	559	Nov. 4, 3-4 P.M.	66,900	120	-	Dam; $C=3.90$.
445	445	- -	45,000	101	-	Rating curve.
809	809	- -	62,900	78	-	Dam; $C=4.00$.
150	150	- -	3,660	24.4	-	Dam; $C=3.30$.
58	58	- -	8,700	167	-	Dam; $C=3.8$. ¹

TABLE 5—FLOOD PROFILES

Winooski River Flood Profile

LOCATION	Miles Above Mouth	Drainage Area (Square Miles)	Fixed Crest (Elevation)	Normal Water Surface (Elevation)	Elevation, 1927 Flood
Lake Champlain	0	1,076	—	97.0	—
Winooski:					
Tailwater, American Woolen Com- pany	9.3	1,058	—	102.0	120
Dam	9.5	1,058	137.0	137.0	158
Tailwater, American Woolen Com- pany	9.8	1,058	—	139.0	158
Dam	9.8	1,058	153.9	153.9	172
Tailwater, Green Mountain Power Corporation	10.5	1,055	—	156.0	173
Dam	10.6	1,055	185.0	189.0	212
Limekiln Gorge	11.7	1,055	—	192.0	240
Essex Junction:					
Tailwater, Green Mountain Power Corporation	16.8	1,018	—	212.0	242
Dam	17.0	1,018	270.3	272.3	289
Huntington River, Mouth	33.1	946	—	309.0	328
North Duxbury:					
Tailwater, Green Mountain Power Corporation	39.5	898	—	339.0	371
Dam	39.8	898	389.0	389.0	427*
Waterbury River, Mouth	41.9	836	—	398.0	434
Mad River, Mouth	48.6	675	—	435.0	465
Middlesex:					
Foot of gorge	48.9	531	—	440.0	467
Dam	49.0	531	—	490.0	515±
Montpelier, U. S. G. S. Gage . . .	56.0	420	—	505.0	527
Montpelier:					
Tailwater, United States Clothes- pin Company	56.1	338	—	505.4	534
Dam	56.1	338	519.8	519.8	538
Tailwater	57.4	336	—	522.0	540
Dam	57.4	336	527.7	527.7	543

* Five hundred feet upstream.

TABLE 5—FLOOD PROFILES—*Continued*
Winooski River Flood Profile—Concluded

LOCATION	Miles Above Mouth	Drainage Area (Square Miles)	Fixed Crest (Elevation)	Normal Water Surface (Elevation)	Elevation, 1927 Flood
Barre:					
Tailwater	61.2	88	—	556.0	568
Dam	61.2	88	562.7	562.7	572
Tailwater	61.5	87	—	564.0	575
Dam	61.6	87	574.8	574.8	587
Tailwater	62.5	86	—	578.5	594
Dam	62.6	86	586.0	586.0	600
Tailwater	63.7	79	—	595.5	—
Dam	63.7	79	607.1	607.1	—

Lamoille River Flood Profile

[Elevations refer to Mean Sea Level]

Lake Champlain	0	710	—	96.3	—
West Milton, highway bridge	5.4	704	—	97.0	117.5
Milton:					
Tailwater, Public Electric Light Company	8.6	694	—	145.8	166.0
Dam	9.0	694	243.4	243.4	261.0
Fairfax Falls:					
Tailwater, Public Electric Light Company	20.9	529	—	339.8	365.0
Dam	21.0	529	421.9	423.4	437.0
Jeffersonville, highway bridge (high- est)	34.4	456	—	444.0	463.5
Johnson, highway bridge	46.4	290	—	479.5	517.5
Morrisville:					
Tailwater, Municipal Power Plant	54.4	250	—	539.0	562.0
Dam	54.7	250	576.1	579.0	590.0
Tailwater, Municipal Power Plant	56.0	230	—	595.0	625.0
Dam	56.1	230	626.8	628.8	639.0

TABLE 5—FLOOD PROFILES—*Continued**Missisquoi River Flood Profile*

LOCATION	Miles Above Mouth	Drainage Area (Square Miles)	Fixed Crest (Elevation)	Normal Water Surface (Elevation)	Elevation, 1927 Flood
Lake Champlain	0	885	—	96.3	102.0
Swanton:					
Tailwater	7.6	847	—	96.9	118.5
Dam	7.7	847	108.1	108.1	123.0
Highgate:					
Tailwater, Village of Swanton . .	14.2	821	—	108.6	146.0
Dam	14.5	821	164.8	167.8	182.0
East Highgate:					
Tailwater, Rixford Manufacturing Company	18.4	815	—	195.2	215.0
Dam	18.5	815	204.2	204.2	221.0
Sheldon Springs:					
Tailwater, Missisquoi Pulp and Paper Company	22.0	806	—	247.1	285.0
Dam	22.5	806	326.0	326.0	340.0
Black Creek, Mouth	25.1	795	—	326.5	348.0
Enosburg Falls:					
Tailwater, L. L. Marsh, Village of Enosburg	33.3	596	—	366.4	395.0
Dam	33.5	596	384.5	386.2	401.0
Samsonville:					
Tailwater, Atlas Plywood Com- pany	38.1	572	—	392.7	416.0
Dam	38.2	572	401.4	402.9	417.5
Richford:					
Tailwater, Public Utilities, Ver- mont Corporation	46.6	480	—	430.0	450.0
Dam	46.7	480	442.2	442.2	457.0
North Troy:					
Tailwater, Blair Veneer Company	67.0	137	—	517.1	540.0
Dam	67.0	137	536.7	536.7	550.0
Phelps Falls:					
Tailwater, Blair Veneer Company	74.5	97	—	682.0	—
Dam	74.6	97	736.2	736.2	—

TABLE 5—FLOOD PROFILES—*Continued**Otter Creek Flood Profile*

LOCATION	Miles Above Mouth	Drainage Area (Square Miles)	Fixed Crest (Elevation)	Normal Water Surface (Elevation)	Elevation, 1927 Flood
Lake Champlain	0	943	—	94.7	99.6
Vergennes:					
Tailwater	7.5	874	—	95.1	103.0
Dam	7.5	874	132.7	133.0	139.0
Lemon Fair River, Mouth . . .	16.2	848	—	134.0	146.2
Weybridge:					
Tailwater	19.8	754	—	143.3	150.0
Dam	19.8	754	161.2	165.2	170.8
Huntington Falls:					
Tailwater	21.2	753	—	175.5	185.0
Dam	21.2	753	214.5	217.5	223.5
New Haven River, Mouth . . .	22.4	751	—	220.0	228.0
Beldens:					
Tailwater	23.2	635	—	241.1	254.0
Dam	23.3	635	281.8	282.8	289.0
Middlebury:					
Tailwater	25.1	631	—	281.8	293.0
Dam	25.3	631	312.4	312.4	319.5
Tailwater	26.2	630	—	313.2	322.0
Dam	26.2	630	336.2	336.2	347.0
Middlebury River, Mouth . . .	30.0	620	—	339.0	351.5
Proctor:					
Tailwater	63.6	364	—	352.9	373.0
Dam	63.7	364	466.8	469.3	487.0
Clarendon Creek, Mouth . . .	70.5	357	—	470.0	494.0
Center Rutland:					
Tailwater	70.9	308	—	480.6	496.0
Dam	70.9	308	507.1	507.1	515.5
Tailwater	71.45	306	—	509.4	518.5
Dam	71.7	306	520.8	520.8	530.0
Cold River, Mouth	74.8	236	—	521.3	535.0
Mill River, Mouth	82.0	182	—	532.0	547.5
Wallingford:					
Tailwater	85.5	106	—	555.9	567.0
Dam	85.6	106	566.1	566.1	578.0
South Wallingford Railroad Bridge	91.3	85	—	587.0	600.0

TABLE 5 — FLOOD PROFILES — Continued
Connecticut River Flood Profile

LOCATION	Miles Above Mouth	Drainage Area (Square Miles)	Fixed Crest (Local Datum)	Local ± U. S. G. S. El.	Normal Water Sur- face El. U. S. G. S.	EL. 1917 Flood		Notes
						Local Datum	U. S. G. S. Datum	
Hartford	48	—	—	+1.16	—	28.3	29.5	—
Windsor Locks	60	—	—	+1.16	—	—	—	—
Enfield Dam	63	9,400	41.01	+1.16	—	—	—	—
Springfield	72	—	—	—	—	—	—	—
Hampden Bridge	76	—	—	—	—	63.1	—	—
Holyoke Bridge	80	—	—	—	—	—	—	—
Holyoke Dam	82	8,390	100.00	-0.87	—	114.75	113.88	—
Sunderland Bridge	107	8,000	—	—	—	34.1	—	—
Mouth, Deerfield River	116	—	—	—	—	—	—	—
Cabot Plant, Turners Falls	117	—	—	—	113.5	74.34	144.34	—
No. 1 Plant, Turners Falls	119	—	—	—	130.5	78.6	148.6	—
Turners Falls Dam	120	6,900	103.00	+70.05	—	113.6	183.6	—
Vernon, tailwater	139	—	—	+93.41	183.5	123.7	217.1	—
Vernon, headwater	139	6,300	118.00	+93.41	217.5	134.8	228.2	—
Brattleboro	145	—	—	+93.41	218.0	137.5	230.9	—
Suspension Bridge	147	—	—	+93.41	—	141.1	234.5	—
East Putney Ferry	157	—	—	+93.41	—	154.8	248.2	—
Walpole Bridge	166	—	—	+93.41	—	161.5	254.9	—
Bellows Falls, tailwater	170	—	—	+178.41	227.0	82.1	260.5	—
Bellows Falls, headwater	170	5,400	100.00	+178.41	291.0	125.7	304.1	—
Cheshire Toll Bridge	180	—	—	+178.41	291.5	131.4	309.8	—
Acutney Bridge	192	—	—	+178.41	292.0	145.0	323.4	—
Windsor Toll Bridge	197	—	—	+178.41	295.0	149.1	327.5	—

Sumners Falls, foot	204	—	—	307.0	—	336.2	—
Sumners Falls, head	204	—	—	315.0	—	336.9	—
Ottaquechee	206	—	—	317.0	—	344.7	—
White River railroad bridge	212	4,120	—	326.0	—	358.5	—
Wilder Dam, tailwater	214	—	—	331.3	—	359.4	—
Wilder Dam, headwater	214	3,400	187.90	368.0	—	378.4	—
Hanover Bridge	216	—	—	368.0	—	384.5	—
Pompanoosuc River	221	—	—	368.0	—	390.2	—
Thetford Bridge	226	—	—	368.5	—	394.2	—
North Thetford Bridge	229	—	—	368.8	—	396.2	—
Orford Highway Bridge	235	3,100	—	370.0	—	401.4	—
Piermont Bridge	242	—	—	373.0	—	407.5	—
South Newbury Bridge	250	2,830	—	377.0	—	410.4	—
North Haverhill Bridge	253	—	—	379.0±	—	411.0±	—
Wells River, Boston & Maine Bridge	263	2,720	—	402.5	—	421.5	—
East Ryegate Dam, tailwater	268	—	—	406.6	—	435.5	—
East Ryegate Dam, headwater	268	2,225	421.30	421.4	—	436.9	—
McIndoes Falls, foot	272	—	—	421.5	—	441.5	—
McIndoes Falls, head	272	—	—	430.2	—	450.1	—
Barnet Bridge (200 feet north)	275	2,200	—	441.0	—	461.4	—
Passumpsic River	277	—	—	456.4	—	471.5	—
Foot of falls	278	—	—	478.0	—	491.5	—
Waterford Bridge	287	1,600	—	655.5	—	664.8	—
Gilman Dam, tailwater	298	—	—	809.0	—	820.0	—
Gilman Dam, headwater	298	1,540	828.10	831.1	—	838.3	—
Lancaster Highway Bridge	310	1,290	—	—	—	848.0	—

Bridge destroyed

TABLE 5—FLOOD PROFILES—*Concluded*
Merrimack River Flood Profile

LOCATION	Miles Above Newburyport Light	Drainage Area (Square Miles)	Fixed Crest (Local Datum)	Local Datum $\pm$ "X" = U. S. G. S. El.	Normal Water Surface Elevation, U. S. G. S.	1896 Flood		1927 Flood	
						Local Datum	U. S. G. S. Datum	Local Datum	U. S. G. S. Datum
Lawrence, Mass. (Essex Company):									
Lower Weir (tailwater gage)	27.0	—	—	—	12.0	29.78	34.82	25.60	30.64
Lawrence Dam	28.0	4,663	34.12	+5.04	44.0	43.98	49.02	43.46	48.50
Lowell, Mass. (Locks and Canals):									
Boott Gage (tailwater)	38.0	—	—	—	49.0	63.11	68.33	59.11	64.33
Pawtucket Dam	40.0	4,097	82.00	+5.22	92.0	94.80	100.02	92.66	97.88
Nashua, N. H. (Nashua Manufacturing Company, Jackson Mills):									
Canal Street Bridge over Nashua River	53.0	3,975	—	+54.86	93.0	63.70	118.56	57.80	112.66
Manchester, N. H. (Amoskeag Manufacturing Company):									
Lower weir (tailwater)	71.0	—	—	—	118.0	37.00	138.10	33.30	134.40
Amoskeag Dam	72.5	2,839	70 & 72	+101.10	175.0	80.95	182.05	81.20	182.30

[illegible]

FLOOD DAMAGE FROM STORM OF NOVEMBER 3-4, 1927

General Description

The flood due to the great storm of November 3-4, 1927, and the subsequent run-off from uncontrolled drainage areas in New England, approached for the first time, perhaps, a national calamity. In importance, it approximated the more frequent flood disasters of the Mississippi and Ohio Rivers. While municipalities, railroads and other structures were generally supposed to be beyond the reach of floods, the rainfall of November, 1927, coming after large rainfall during October, very suddenly produced flood conditions entirely unexpected and



PLATE III. — CANADIAN NATIONAL RAILWAY BRIDGE, GORHAM,
N. H., ON PEABODY RIVER, NOVEMBER, 1927

far beyond the extent thought possible, resulting in devastation and terror wholly without precedent in many localities.

Entire river valleys were flooded, 87 lives lost, rail and highway transportation brought to a standstill, and many homes, industries and business centers destroyed or badly damaged.

Those river valleys in Vermont in which the most serious flood damage occurred — the Winooski, White, Black, Lamoille and Otter Creek — are narrow with steep hills, mostly wooded, on each side. The widths of the valley bottoms, which accommodate the river, the highway and usually a railroad, vary from the width of the river alone, with road and railway cut into the sidehill, to nearly a mile in some places. The towns that suffered most severely were located on these

narrow flood plains, which were entirely covered with water at the crest of the flood. The relatively steep slopes of these valleys caused very high velocities, especially at constricted sections, which helps to explain the great destruction of property.

The damage due to the November, 1927, flood, in point of lives and property lost beyond replacement, was largely in Vermont, with considerable damage in portions of New Hampshire and western Massachusetts.

The property losses in Vermont from the November flood aggregated over \$26,000,000, divided as follows:

690 farms	\$1,350,000	
Industrial establishments	5,558,800	
137 cities and villages	6,121,151	
Steam and electric railroads (other than Rutland and Central Vermont)	1,151,200	
Vermont State Hospital	400,000	
Telephone and telegraph companies	319,050	
Gas companies	30,400	
Boston & Maine Railroad	640,000	
Rutland Railroad Company	750,000	
Central Vermont Railroad	2,750,000	
		\$19,070,601
Federal aid roads	\$2,599,872	
State aid systems	2,577,114	
Local roads and bridges	1,886,012	
		7,062,998
		\$26,133,599

The United States Engineer Corps has made a segregation of flood losses on certain of the river systems of Vermont, upon which Table 6, is based:

TABLE 6. — FLOOD DAMAGE IN VERMONT, 1927 — BY RIVER SYSTEMS

RIVER SYSTEM	Farms	Highways and Bridges	Railroads	Industrial	Cities and Towns	Economic	Total
Winooski .	\$200,000	\$1,396,000	\$1,590,000	\$3,500,000	\$4,100,000	\$2,435,000	\$13,221,000
Lamoille .	100,000	525,000	210,000	160,000	410,000	170,000	1,575,000
Missisquoi .	120,000	570,000	190,000	300,000	370,000	245,000	1,795,000
Otter Creek .	135,000	535,000	760,000	1,370,000	260,000	840,000	3,900,000
Total .	\$555,000	\$3,026,000	\$2,750,000	\$5,330,000	\$5,140,000	\$3,690,000	\$20,491,000

In the region of greatest damage — the Winooski River, with a drainage area of about 1,000 square miles — the damage per square mile was therefore about \$13,200, or about \$20 per acre.

To understand the significance of these figures the losses should be compared with the population and wealth of the state, which are given in Table 7. On a per capita basis, the flood damage in Vermont represented about \$74 per person, a very considerable burden upon its inhabitants. The most authentic figure of total damage to New England property of all kinds in November, 1927, is about \$40,000,000, divided approximately as shown in Table 7.

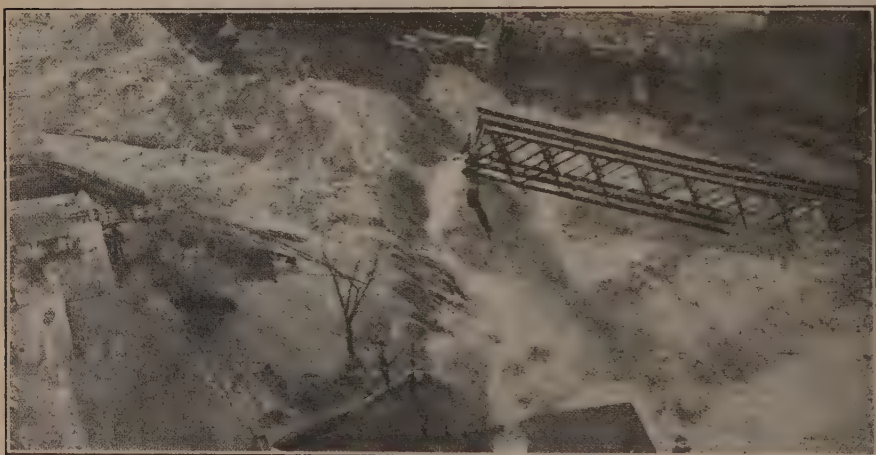


PLATE IV. — GORGE ON OTTER CREEK, RUTLAND, VT., NOVEMBER, 1927

For the time being the loss of life overshadowed other damages and losses. The speedy response by the Red Cross, Federal, State, and local authorities, the colleges, New England Council, and banking institutions, coupled with the wonderful spirit shown by those living within the flooded district, helped greatly to mitigate the calamity, resulting in an instant co-operation of effort not possible under normal conditions.

Immediate relief in the form of the transportation of workers and necessities of life into the flooded districts was furnished by trucks as soon as roads became at all passable, and most of the railroads, except the Central Vermont, were giving partial service in less than three weeks. Service on the main line of the Central Vermont was not re-established until three months after the flood, while some of the branch

TABLE 7. — POPULATION, WEALTH AND FLOOD DAMAGE — NEW ENGLAND STATES

LOCATION	Areas (Square Miles)	Per Cent of Total	Popu- lation, 1920*	Per Cent of Total	Wealth, 1920 (Taxable) *	Per Cent of Total	Total Flood Damage as Reported †	Per Cent of Total	FLOOD DAMAGE	
									As Per Cent of Taxable Wealth	Per Capita
Vermont	9,564	14.40	352,428	4.76	\$799,000,000	3.54	\$26,133,000	65.94	3.27	\$74.30
New Hampshire	9,341	14.07	443,083	5.99	1,283,000,000	5.69	6,400,000	16.15	.49	14.50
Massachusetts	8,266	12.44	3,852,356	52.05	11,895,000,000	52.75	6,000,000	15.14	.05	1.55
Connecticut	4,965	7.47	1,380,631	18.65	4,842,000,000	21.47	1,000,000	2.52	.02	.72
Maine	33,040	49.74	768,014	10.38	1,919,000,000	8.51	100,000	.25	.005	.13
Rhode Island	1,248	1.88	604,397	8.17	1,814,000,000	8.04	0	—	—	—
Total	66,424	100.00	7,400,909	100.00	\$22,552,000,000	100.00	\$39,633,000	100.00	.175	\$5.35

* Statistical Abstract of the United States, United States Department of Commerce, 1923.

† "Boston Herald," January 12, 1928.

lines were out of commission a much longer time. Permanent reconstruction of highways, bridges and damaged property was not completed until the following year.

The total loss of life caused by the flood of November, 1927, in New England was 87, of which 84 occurred in Vermont and 3 in Massachusetts.



PLATE V. — MAIN STREET, MONTPELIER, VT., NOVEMBER, 1927

Following is a summary for Vermont with reference to river valleys:

LOSS OF LIFE IN VERMONT, NOVEMBER, 1927, FLOOD

LOCATION	River Valley	Deaths in Flood	Deaths Due to Causes Incident to Flood	Total
West side of state	Winooski . . .	55	3	58
	Lamoille . . .	4	—	4
	Otter Creek . . .	1	5	6
	Missisquoi . . .	2	—	2
	Walloomsac . . .	1	—	1
East side of state . . .	Clyde . . .	1	—	1
	Connecticut . . .	—	1	1
	Black (North) . . .	2	—	2
	White . . .	9	—	9
		75	9	84

In Massachusetts the loss of one life was due to the break of the dam at Becket, and two motorists who attempted to follow the state highway between Springfield and Westfield were drowned.

A map showing the different river valleys and the localities where human lives were lost, with their number, is almost an index map to the location of the damages of all kinds. The Winooski River from Barre to Burlington, particularly in the towns of Waterbury and Bolton, and the White River from Bethel to White River Junction, rose with great rapidity, far beyond any previous knowledge, before the inhabitants realized what was taking place. The sudden rise of water, the carrying away of bridges, the piling up of débris, and the cutting of new stream channels caused conditions thought impossible heretofore.

The greatest losses of life occurred in the following towns, all within the Winooski River Basin:

Bolton	.	.	.	.	.	.	.	.	.	.	26
Waterbury	.	.	.	.	.	.	.	.	.	.	11
Barre	.	.	.	.	.	.	.	.	.	.	7
Moretown	.	.	.	.	.	.	.	.	.	.	5
Duxbury	.	.	.	.	.	.	.	.	.	.	5

Property damages are classified as follows: dams, railroads, highways and flood areas, and are outlined below.

Dams and Abutments

There was no instance of the failure of any dam of importance on the principal rivers of New England during the flood of November, 1927. This applies to the rivers of Vermont, the Connecticut River, Merrimack River and tributary streams. The dams which were washed out or undermined were across small tributary streams in the upper reaches of the valleys. This is particularly true of the Winooski River on the west side of Vermont, and the White and Passumpsic Rivers on the east side.

The old wooden dam, 48 feet high and 155 feet long, on the Winooski River at Middlesex, Vt., was carried away. This was in poor condition and was scheduled for early replacement. It controlled only a very small pond.

The dam at Bristol, N. H., was not damaged, but quick work with sand bags was necessary in order to prevent cutting around one end of it.

The situation at Bellows Falls, where construction was proceeding, was serious for a while, and much damage was done to the temporary

structures and to the railroad and other property there. The large dams on the Connecticut and Merrimack Rivers suffered no damage.

A partial list of dams and abutments in Vermont washed out or damaged includes the following:

<i>West Side</i>									
Winooski River	.	.	.	.	.	.	.	.	24
Lamoille River	.	.	.	.	.	.	.	.	2
Missisquoi River	.	.	.	.	.	.	.	.	1
Broad Brook	.	.	.	.	.	.	.	.	1
Otter Creek	.	.	.	.	.	.	.	.	1

<i>East Side</i>									
White River	.	.	.	.	.	.	.	.	11
Passumpsic	.	.	.	.	.	.	.	.	7
Black River (North 1)	}	.	.	.	.	.	.	.	3
Black River (South 2)		.	.	.	.	.	.	.	
Ottaquechee River	.	.	.	.	.	.	.	.	3
West River	.	.	.	.	.	.	.	.	3
Barton River	.	.	.	.	.	.	.	.	2
Wells River	.	.	.	.	.	.	.	.	2
Clyde River	.	.	.	.	.	.	.	.	1
Connecticut River	.	.	.	.	.	.	.	.	1
Total	.	.	.	.	.	.	.	.	62

Those in New Hampshire included:

Pemigewasset River	.	.	.	.	.	.	.	.	4
Smith River	.	.	.	.	.	.	.	.	1
Mad River	.	.	.	.	.	.	.	.	2
Ammonoosuc River	.	.	.	.	.	.	.	.	1
Total	.	.	.	.	.	.	.	.	8

The following is the list of dams injured in Massachusetts as given by the Department of Public Works:

Blackstone River	.	.	.	.	.	.	.	.	3
Westfield River	.	.	.	.	.	.	.	.	1
Total	.	.	.	.	.	.	.	.	4

Railroads

The extensive damage to railroads is fully presented in the report of the Special Committee of the American Railway Engineering Association.*

* "New England Flood of November, 1927," Bulletin of the American Railway Association, August, 1928.

The summary as given in that report, with the addition of the river systems, follows:

LOSS AND DAMAGE TO RAILROADS, NEW ENGLAND FLOOD OF NOVEMBER, 1927

RIVER	DIRECT LOSSES	
	Railroad	Property Damage
Aroostook	Bangor & Aroostook	\$4,000
Westfield	Boston & Albany	350,000
Hoosac	Boston & Maine	2,500,000
Pemigewasset		
Connecticut		
Merrimack		
Saco		
Passumpsic	Canadian Pacific	1,250,000
Missisquoi		
White, Winooski		
Otter Creek		
Saco		
Winooski, Wells	Central Vermont	2,750,000
Connecticut	Delaware & Hudson	283,000
Otter Creek	Maine Central	200,000
Black	Montpelier & Wells River	190,000
Walloomsac	New York, New Haven & Hartford	100,000
Winooski	Rutland	750,000
Lamoille		
	St. Johnsbury & Lake Champlain	291,000
Total		\$8,668,000
Traffic, operating and miscellaneous losses (not complete)		4,131,000
Grand total		\$12,799,000

Highways

The highways suffered as did the railroads, and in about the same localities. Most of the damage in New England was confined to the areas flooded by the Winooski, Lamoille, Missisquoi and Otter Creek on the west side of Vermont, and the White, Black and Passumpsic on the east side; the valleys of the Ammonoosuc, Israel and main upper Connecticut, and the Baker, Mad and other tributaries of the Pemigewasset, in New Hampshire. As in the case of the dams, most of the

bridges carried away and roads washed out were on the tributaries. On the main Connecticut and Merrimack rivers little damage resulted.

The damage to highways and bridges in the different states was as follows:

*Vermont.** — There were 1,214 bridges damaged or destroyed (structures over 4 feet span); 9 over 200 feet; 11 over 150 feet; 27 between 100 and 150 feet.

The estimated cost of replacement was:

Bridges	\$4,579,082
Highways	2,483,916
	\$7,062,998

The estimated damage to roads and bridges by river systems was:

White River	\$1,500,000
Winooski River	1,250,000
Lamoille River	640,000
Otter Creek	580,000
Passumpsic River	360,000

New Hampshire. — A summary of the damage to New Hampshire highway bridges (of over 10 feet span) is as follows:

RIVER	Number of Bridges Damaged or Destroyed
Ammonoosuc	18
Wild Ammonoosuc	2
Connecticut	2
Oliverian Brook	3
Gale, Israel and Sugar	4
	29
Pemigewasset	16
Androscoggin, Peabody, Saco, etc.	7
Miscellaneous	2
	25
Total	54

* "Reconstruction of Vermont Highways," by Stoddard B. Bates, Commissioner of Highways, Journal, Boston Society of Civil Engineers, December, 1928.

Massachusetts. — The losses to highways and highway bridges in Massachusetts were not great. Details of bridges and highway replacement estimates classified by rivers were as follows: *

RIVER	ESTIMATED COST	
	Bridges	Highways
Hoosac	\$106,900	\$114,950
Housatonic	10,675	42,868
Westfield	33,900	44,575
Deerfield	2,450	26,850
Farmington	1,200	6,150
Hudson	—	4,900
Connecticut	6,650	28,810
Swift	—	3,500
Chicopee	500	2,100
Millers	—	7,150
Nashua	13,107	77,178
Concord	56,033	85,999
Blackstone	46,035	103,383
Ware	—	4,150
Quaboag	—	1,000
French	—	1,100
Shawsheen	—	600
Charles	—	1,000
Merrimack	—	18,680
Totals	\$277,450	\$574,943

Connecticut. — Only in a few instances was there any considerable damage to bridges and highways.

* District Engineers' Reports of Flood Damage, Massachusetts Department of Public Works.

Flood Areas

The record for Vermont of houses, mills and business districts affected includes, as a partial list:

RIVER VALLEY	Houses	Mills	Business Districts
Winooski	552	153	80
Lamoille	47	7	16
Missisquoi	8	4	—
Otter Creek	100	4	—
Black River (North)	2	3	3
Clyde	—	2	1
Barton	2	3	2
Passumpsic	17	14	6
Wells	—	5	1
West	—	3	1
White	62	11	15
Ottauquechee	102	6	1
Black River (South)	35	7	7
Connecticut	—	4	—
Totals	927	226	133

Of the larger business centers, Montpelier and Barre were most seriously affected. While the business section of Montpelier was under 10 feet or more of water with a swift current, it is to be noted that but one life was lost in this city, as most of the flooded buildings were of substantial construction and were not carried away or destroyed. In the flooded areas many farms and country places were destroyed, including farm buildings and stock. A considerable amount of bottom lands was also ruined by deep deposits of sand and gravel. In individual cases these losses can never be made good.

SUMMARY

The loss of human life and property in Vermont was appalling, but this was tempered by the instant response of every source of help, — neighbors, State, New England, Federal, Red Cross, banking houses and others.

The railroad losses in many cases almost wiped out years of improvement in roadbed and bridges. The Central Vermont Railroad

was forced into receivership, and the St. Johnsbury and Lake Champlain Railroad required financial assistance from the State.

While the flood was a great calamity for Vermont, and the loss of life deplorable, most of the scars left in its wake are fast disappearing, and its railroads, highway systems and bridges are already restored and rebuilt in better condition than ever before.

Chapter III. — Previous Great Storms in New England

A study of great storms in New England indicates the following chronological list of exceptional storms or occasions of especially noteworthy rainfall, beginning with that of 1927. Rainfall-area relations for these storms when available are given in Table 8.

EXCEPTIONAL STORMS IN NEW ENGLAND, 1770 TO 1927

- | | |
|-------------------------------|--------------------------|
| 1. November 3-4, 1927. | 9. July 24-25, 1830. |
| 2. September 24-29, 1909. | 10. August 28, 1826. |
| 3. July 12-14, 1897. | 11. March 24-25, 1826. |
| 4. February 29-March 4, 1896. | 12. March 5-6, 1823. |
| 5. October 12-14, 1895. | 13. May 13-19, 1814. |
| 6. February 11-14, 1886. | 14. March 18-22, 1801. |
| 7. October 3-4, 1869. | 15. October 20-22, 1785. |
| 8. April 27-May 1, 1854. | 16. January 7-8, 1770. |

September 24-29, 1909. — This 4-day storm centered in the State of Maine. No noteworthy damage occurred from resulting high water, as this storm followed a three months' period of drought.

July 13-14, 1897. — This storm was a memorable and destructive one in Connecticut and western Massachusetts, lasting from 30 to 36 hours. This storm came from the West, moving across the Great Lakes on the 12th and centering in western New England on the 13th and 14th, resulting in precipitation varying from 5 to 9 inches. Southington, Conn., 10.3 inches, Bridgeport, 9.39 inches, and Hartford, 9.22 inches, were in the area of greatest rainfall. In western Massachusetts, Vermont, New Hampshire and Maine the amounts also reached 5 or 6 inches, but rainfall along the coast was small.

February 29-March 4, 1896. — This heavy rain, with heaviest fall in Maine, caused, with melting snow, severe floods and attendant damage. The Merrimack River at Lawrence reached the highest stage during its long time of record.

October 12-14, 1895. — This began in southwestern New England at midnight October 11, in eastern Massachusetts about 2 P.M. of October 12, and in southwestern Maine later that afternoon. It ended in eastern Massachusetts by 4 A.M. of the 14th, and in northern New England before midnight of the 13th, thus lasting about 36 hours. The heaviest rainfall occurred in an area between northeastern Connecticut and Boston, with maximum amounts reaching nearly to those of November, 1927. Owing to a previous very dry period, no serious floods occurred, even in the area of greatest rainfall, and no marked damage resulted.

February 11-14, 1886. — At this time occurred a rainstorm with the added effect of deep snow, which by melting contributed about 2 inches of water, causing the greatest freshets in southeastern New England in the 40 years prior to 1927. It passed northeasterly, diagonally across Rhode Island from the Connecticut boundary and across Massachusetts to the sea, the maximum rainfall of 5.65 inches occurring in the vicinity of Providence, R. I. It began early on the 11th and ended on the afternoon of the 13th, lasting about 60 hours.

At the beginning the temperature was low, the rain was largely absorbed by the snow, with little run-off. On the night of the 12th and morning of the 13th, however, the temperature rose to above 50°, at which time, also, the bulk of the rainfall occurred, resulting in great freshet effect. Stony Brook in Boston produced an extraordinary flood, inundating a thickly populated district, and great damage resulted in many other cities and towns in southeastern New England. The records of this flood have furnished a basis for the design of waterways and other structures in this region since that time.

October 3-4, 1869. — One of the greatest freshets that has ever occurred in New England resulted from this storm, which is proverbially known as the "Punkin Freshet." The storm lasted about 36 hours and reached a maximum rainfall of 12.35 inches at Canton, Conn., covering practically all of New England and extending as far south as Virginia.

April 27-May 1, 1854. — This was a 90-hour storm with a total rainfall said to be 7 inches at Hartford, Conn., resulting in the highest recorded stage of the Connecticut River at Hartford, viz., 29.8 feet. It is also said that about a foot of wet snow fell the previous week in the headwaters of the Connecticut which melted at this time.

July 24-25, 1830. — A great rainfall occurred at this time, centering in Vermont, resulting in a remarkable freshet. Up to the middle of July the summer had been very cold and wet. A week of great heat followed, and on the evening of July 24 the storm began, continuing

during that night and the next day. For three more days there were frequent heavy showers, and at Burlington 7 inches of rain fell during the 5 days, 3.85 inches of this on the 26th in 16 hours.

The resulting floods in Vermont on Otter Creek, the White, Winoo-ski and Passumpsic Rivers had many of the features of the 1927 flood, many bridges, buildings and other structures being washed away, and many deaths resulted. A flood mark on a boulder near the Pittsford Railroad Station, said to have been made at the time of this flood, showed a water level only a foot lower than that of the 1927 flood, at this place.

August 28, 1826. — The month of August in southern New England was one of the wettest ever experienced. At Salem, Mass., 14 inches of rain fell during the month, with 8.7 inches from the 10th to the 15th, including 2.5 inches on the 14th.

In the White Mountains the month was very dry until the 28th, when a torrential rain of about 24 hours' duration occurred, causing one of the most remarkable floods which has ever taken place in this vicinity, with much damage. This was the storm in which the famous "Willey House slide" occurred.

March 24-25, 1826. — This was an unusual storm and freshet in northern New England, extending into New York and Canada. Rain began on the evening of the 24th, with a southeast gale, and extended into the next morning. The rainfall was torrential, but no records are available as to its amount.

In Vermont much damage was done to roads, bridges and buildings, and some loss of life occurred. Montpelier was almost entirely inundated and much loss of farm stock resulted.

This storm also caused very serious ice jam floods on the Kennebec River, in Maine.

March 5-6, 1823. — A disastrous freshet occurred in Rhode Island and Connecticut, due to a very heavy 24-hour rainfall at a time when deep snow prevailed. Many bridges and buildings were carried away.

May 13-19, 1814. — The month of May, 1814, in the State of Maine was very wet, culminating in a heavy storm lasting 4 days, with some further rain during 2 more days. The resulting freshet was characterized as the greatest in 30 years, with resulting loss of many bridges, mills, houses and logs, particularly on the Androscoggin and Saco Rivers.

March 18-22, 1801. — This was a 4-day rainstorm causing a great flood, particularly in southeastern Vermont, Massachusetts and Connecticut, with much damage to mills, bridges, houses and other property, and the loss of several lives.

October 20-22, 1785. — The month of September was very wet, this continuing until October 20, when in the evening the wind shifted from northwest to southeast, continuing thus with very heavy rainfall, totaling 9 inches in the 3 days. It centered in southeastern New Hampshire and resulted in great freshets on the Merrimack and other rivers in New Hampshire. In southwestern Maine, on the Saco and Monson Rivers, much damage resulted.

January 7-8, 1770. — Beginning at 1 A.M. of January 7 this great rainstorm lasted until noon of the 8th. The most damage occurred on the Kennebec and Androscoggin Rivers, in Maine, with the loss of many mills and dams, owing particularly to heavy ice going out. The Connecticut River also did great damage.

The areas as given in Table 8 are limited to New England and eastern New York, and do not, in the case of the great storms of 1869 and 1897, include areas covered in the Middle Atlantic States farther south. Thus in the 1869 storm the total area covered by the 6-inch rainfall line was about 24,000 square miles, of which 13,570 (as given in the table) was in or near New England.

TABLE 8. — SUMMARY OF RAINFALL-AREA RELATIONS FOR NEW ENGLAND

[Area (square miles) covered by rainfall in inches]

DATE OF STORM	RAINFALL (INCHES)								Maximum Re- corded
	5	6	7	8	9	10	11	12	
Oct. 3-4, 1869	—	13,570	7,150	2,260	902	370	255	40	12.35
Feb. 11-14, 1886	4,850	3,000	1,500	750	—	—	—	—	—
Oct. 12-14, 1895	6,340	4,070	2,040	590	—	—	—	—	—
Feb. 29-Mar. 4, 1896	5,920	3,495	2,350	1,485	465	—	—	—	9.54
July 13-14, 1897	11,000	4,230	2,550	1,620	550	60	—	—	10.30
Nov. 3-4, 1927	22,030	8,470	4,040	1,795	497	—	—	—	9.65

ACCURACY OF RECORDS

Data for the foregoing storms subsequent to and including that of 1869 are fairly complete and accurate. In general, for the earlier storms no detailed rainfall data are available, and their severity must be judged from the statements of resulting freshets and damage. There were other storms during this time that were perhaps of importance, but those listed are apparently the outstanding ones in this period of about 160 years.

Chapter IV. — Flood Factors for New England

INTRODUCTION

The magnitude and frequency of floods are subjects of considerable importance for all streams, large and small, as affecting the design of sewerage systems, bridges, channels required for streams and rivers, spillways and abutments of dams, and the location of roads, railroads and all structures bordering on them.

Probably in most cases the design of water passages and the location of river structures have been based upon local experience. Thus when one bridge has been carried away by a flood, the next one is built a little higher, and so on. This method has certain advantages. It takes into account most of the factors which affect the flood conditions at a particular point. It eliminates excessive costs that would result if river structures were designed to be safe against floods which could never occur, or where the conditions are such that a flood will result solely in temporary inconvenience.

On the other hand, as the valleys become more densely populated, and as economic relations become more interdependent, the results of an excessive flood become more and more disastrous. What at one time would have been simply an inconvenience may under changed conditions become a major catastrophe. This section of the report is devoted to a consideration of the various elements which produce floods and the flow to be expected. Local conditions which determine the height to which the water will rise at any particular point, and the damage which will result, are not considered.

The following are the principal factors involved:

1. Rainfall.
2. Rainfall Run-off Relations.
3. Snow.
4. Drainage Area Characteristics.

RAINFALL

Floods are caused primarily by precipitation. The length of the storm, its intensity, and the area covered are the important factors.

Duration and Intensity of Precipitation

The longest recording rain gage record in New England is that maintained at Chestnut Hill Reservoir in Boston, which has been kept continuously since 1879. Analysis of this 50-year record (1879-1928) has been made,* from which it appears that the relation between duration of rain and average intensity of precipitation at Boston can be expressed by the formula —

$$i = \frac{K}{(t+7)^{0.7}}$$

where i = intensity of precipitation, in inches per hour

t = duration, in minutes

K = a constant, varying with the frequency and represented by the formula $K = 16 F^{0.27}$ (F being frequency in years)

This relation was found to be applicable to durations from 5 minutes to 72 hours, and may probably be applied to any duration of storm likely to be experienced in New England, say, up to 5 or 6 days.

For long storms it will be more convenient to use the cumulative amount of rain rather than intensity, and the period of time in hours rather than minutes. Using P for the cumulative amount of rain and T for time in hours, the formula becomes —

$$P = \frac{.911 T F^{0.27}}{(T+0.117)^{0.7}}$$

For durations greater than, say, 5 hours, T may be used instead of $(T+0.117)$ in the denominator, and the expression becomes —

$$R = 0.911 T^{0.3} F^{0.27}$$

The amounts of rainfall to be equaled or exceeded at Boston as determined by this formula are given in Table 9:

* By Charles W. Sherman, Proceedings, American Society of Civil Engineers, April, 1930.

TABLE 9. — RAINFALL-FREQUENCY RELATIONS

[Based on Chestnut Hill Records]

TIME IN HOURS	FREQUENCY IN YEARS			
	10	25	50	100
1	1.57	2.00	2.42	2.92
2	2.01	2.57	3.10	3.74
3	2.30	2.94	3.55	4.28
5	2.70	3.46	4.18	5.04
8	3.13	4.02	4.84	5.83
12	3.58	4.58	5.52	6.66
24	4.40	5.64	6.80	8.20
36	4.97	6.37	7.68	9.26
48	5.42	6.94	8.37	10.09
60	5.79	7.42	8.95	10.79
72	6.12	7.84	9.45	11.40
96	6.67	8.54	10.30	12.42

Whether these figures are applicable elsewhere in New England, as, for instance, in the Connecticut valley, is not known. Some years ago the recording rain gage record at Springfield was studied, and it was concluded that what was then considered a suitable rain intensity curve for 15-year frequency at Boston would correspond to the curve of 5-year frequency at Springfield, storms longer than 3 hours being left out of consideration in both cases. This indicates a greater intensity of precipitation for short storms at Springfield than at Boston for the same frequency, or more frequent occurrence of storms of equal intensity.

It is probably safe to assume that *at least as great* intensities and frequencies as are given by these formulæ should be expected anywhere in New England.

The records of isolated cases of very heavy precipitation in relatively short intervals at various places in New England indicate that the intensities shown by the Chestnut Hill study have also been equaled at other points. A partial list of such storms is given in Table 10:

TABLE 10. — PARTIAL LIST OF HEAVY RAINSTORMS IN SHORT INTERVALS

LOCATION	Date	Rain- fall in Inches	Remarks
Bridgeport, Conn. . . .	July 29-30, 1905	11.32	Water Supply Paper No. 162, p. 1: Total of 11.32 in. in 17 hrs. 40 min.; 10.08 in. in 6 hrs. 20 min.
Waltham, Mass. . . .	Aug. 21, 1860	5.63	In 3 hrs.
Oxoboxo Brook, near Montville, Conn. . . .	Aug. 25, 1877	8.5	1880 Census of Water Power, Part 1, p. 188: In 3 hrs.
Providence, R. I. . . .	July 12-13, 1834	7.00	Caswell's Weather Record: Total on night of July 12- 13.
West Waterville, Me. . .	Oct. 3-4, 1869	4.10	In 12 hrs.
West Waterville, Me. . .	Oct. 23-24, 1878	6.30	In 19 hrs.
Lewiston, Me. . . .	Mar. 1, 1896	7.36	Preceded by a rain of 1.18 in. on Feb. 29.
St. Johnsbury, Vt. . . .	July 28, 1913	4.99	2 in. in 45 min.
Fall River, Mass. . . .	June 9-10, 1875	4.94	In 12 hrs.
Fall River, Mass. . . .	Jan. 10, 1881	4.60	In 14 hrs.
Framingham, Mass. . . .	Oct. 14, 1895	8.49	— — —
Westborough, Mass. . .	Oct. 12-14, 1895	8.25	In 36 hrs.
Amherst, Mass. . . .	July 13-14, 1887	5.79	5 in. fell in 24 hrs.
Vineyard Haven, Mass. .	Sept. 25-26, 1888	6.00	In 11 hrs.
Nantucket, Mass. . . .	Sept. 10, 1854	6.06	In 2 hrs. 15 min.
Bridgeport, Conn. . . .	July 13, 1897	8.11	— — —
Fort Trumbull, Conn. . .	Aug. 7-8, 1874	9.20	7.30 in. in 6 hrs.; 9.20 in. in 17 hrs.
New Haven, Conn. . . .	Aug. 8, 1874	7.60	4.14 in. additional Aug. 9.
New Haven, Conn. . . .	July 30, 1876	4.50	2.65 in. additional July 31.
Southington, Conn. . . .	July 13-14, 1897	10.30	In 33 hrs.
White Plains, N. Y. . . .	Oct. 3-4, 1877	5.30	In 12 hrs.
White Plains, N. Y. . . .	Oct. 8-9, 1877	9.70	In 10½ hrs.
Lowell, Mass. . . .	July 29, 1885	3.93	In 1½ hrs.

Rainfall-Area Relations

As the size of a watershed increases, the area covered by intense storms becomes of greater importance. In Table 11 are given:

(A) Maximum rainfall-area relations.

(B) Rainfall-area relations likely in any 20-year period.

These figures have been made from a plot and study of the data

in Table 8, and give an approximation of what may be expected, somewhere in New England, (A) as a *maximum* and (B) on the average about once in 20 years.

TABLE 11. — EXPECTED RAINFALL-AREA RELATIONS FOR NEW ENGLAND

	RAINFALL IN INCHES							
	5	6	7	8	9	10	11	12
(A) Maximum area likely to be covered (square miles) . . .	25,000	15,000	8,000	2,500	1,000	500	250	50
(B) Area likely to be covered in any 20-year period (square miles) .	15,000	8,000	4,000	2,000	600	50	—	—

Frequency of Great Storms in New England

Time of Occurrence. — The 16 great storms previously listed in Chapter III occurred in the months of the year, as follows:

January	1	August	1
February	1	September	1
March	4	October	3
April	1	November	1
May	1		
July	2	Total	16

with none occurring in the months of June and December.

Location. — The general location of these storms was as follows:

Northern New England	8
Southern New England	5
General in New England	3
	—
	16

The storms included as general in New England are those of 1869, 1897 and 1927. It is of interest to note that these were all storms of the Atlantic coast type, all occurring either in the summer or fall.

These three are the outstanding great storms in New England in this period of about 160 years.

Frequency. — It will be seen from the foregoing that great storm average frequency has been as follows:

LOCATION IN NEW ENGLAND							Number of Storms in 160 Years	Average Time of Occurrence (Years)
Northern	.	.	.	.	.	.	8	20
Southern	.	.	.	.	.	.	5	32
General	.	.	.	.	.	.	3	53
Total	.	.	.	.	.	.	16	10

The three greatest storms actually occurred within a period of the last 58 years, or one about every 20 years, on the average, in this time. If these three greatest storms are included as occurring in both northern and southern New England, as was the case, this would give —

Northern New England, 11 storms, or one every 15 years.

Southern New England, 8 storms, or one every 20 years.

It may therefore be concluded that, on the average, storms of sufficient magnitude to cause serious flood damage may be expected to occur about every 20 years somewhere in New England, except, perhaps, in the extreme northerly portions. Outstanding or general great storms covering most of New England may be expected every 30 or 40 years. These are *average figures*. No one can foretell when such storms may occur or the actual time intervals between them. They may, of course, occur even in two successive years.

Rainfall Estimates for Maine

Mr. George W. Mindling,* has prepared Table 12, giving his opinion of the 100-year rainfall limits for the State of Maine. These estimated figures would doubtless apply to substantially all of New England. His opinion of the probable absolute maximum rainfall to be expected in Maine is given in Table 13.

* "Maine Rivers and their Protection from Possible Flood Hazards," Maine Development Commission, 1929.

TABLE 12. — ESTIMATED 100-YEAR RAINFALL LIMITS IN MAINE

AMOUNTS OF RAINFALL (INCHES)	LIMITS OF AREA IN SQUARE MILES LIKELY TO BE COVERED WITHIN THE VARIOUS INTERVALS STATED BELOW					
	1 Day	2 Days	4 Days	1 Week	2 Weeks	1 Month
2.00	33,000	—	—	—	—	—
3.00	22,000	33,000	—	—	—	—
4.00	16,000	20,000	25,000	33,000	—	—
5.00	10,000	15,000	20,000	29,000	33,000	—
6.00	2,000	10,000	15,000	19,000	27,000	33,000
7.00	1,500	6,000	10,000	14,000	20,000	28,000
8.00	1,000	3,000	6,000	10,000	15,000	24,000
9.00	500	1,000	4,000	7,000	10,000	20,000
10.00	—	500	2,000	4,000	6,000	16,000
11.00	—	—	1,000	2,000	3,000	12,000
12.00	—	—	—	100	1,000	10,000
13.00	—	—	—	—	600	8,000
14.00	—	—	—	—	300	6,000
15.00	—	—	—	—	100	4,000
16.00	—	—	—	—	—	3,000
17.00	—	—	—	—	—	2,000
18.00	—	—	—	—	—	1,000
19.00	—	—	—	—	—	500
20.00	—	—	—	—	—	100
Approximate state averages in inches . . .	4.10	5.20	6.00	7.00	8.00	10.50

TABLE 13. — MAXIMUM RAINFALL IN MAINE (1,000-YEAR FREQUENCY)

INTERVALS	State Average (Inches)	Range within State (Inches)
24 hours	4.50	3.00- 9.00
2 days	5.50	4.00-10.00
4 days	6.50	4.50-11.00
1 week	8.00	6.00-12.00
2 weeks	10.00	8.00-14.00
1 month	12.00	9.00-20.00

RAINFALL-RUN-OFF RELATIONS

The variations in the percentage of surface run-off due to different conditions of the ground have long been generally recognized. Few quantitative results have been determined. In recent years the belief

has grown that watersheds differ widely from each other in respect to capacity for absorption and retention of rainfall. The Miami Conservancy District's study of this subject concludes —

That the character and condition of the soil influences run-off to a greater extent than has been generally recognized, and next to rainfall is the most important factor affecting run-off.*

Table 14 has been prepared to show the relation between run-off and rainfall for certain storms on certain streams from data believed to be reasonably accurate as to both the rainfall and the run-off. The rainfall stations used to determine the average precipitation in most cases do not include stations on the mountain tops, but in each case a number of stations, well distributed over the drainage area, were available.

From these tables it appears that the per cent of flood run-off on the Swift River above West Ware is distinctly less than for any of the other watersheds. This is reasonable considering the sandy, porous nature of the soil on the Swift River watershed with its flat slopes. It also appears from the tables that the per cent of run-off for the Middle Branch of the Westfield River and for the Miami River is somewhat greater than for the other streams. A comparison of the run-off of the Nepaug River and the East Branch of the Farmington from the storm of November 2-4, 1927, also indicates a considerable difference in the absorptive capacity of these two near-by watersheds.

As will be seen from these tables, during the summer season the per cent of run-off for all the streams varies between a minimum of 1.2 per cent and a maximum of 78.0 per cent, with an average of 26.0 per cent, while during the remainder of the year the per cent of flood run-off varies between a minimum of 14.5 per cent and a maximum of 114.0 per cent, the latter due to melting snow added to the run-off from rainfall, with an average of about 46 per cent.

Great storms such as produce severe floods will generally have a higher per cent of run-off than that given by average figures based largely on moderate storms, as is the case in Table 14. There are exceptions to this, as in almost every one of the watersheds included in Table 14 there is a particular rainstorm of 3 to 4½ inches, with as little as from 1.2 to 20 per cent of run-off. The storm of October 12-14, 1895, did very little damage because soil conditions were such that a large part of the rainfall was absorbed. Excepting under very dry conditions during the summer season it is believed that rainstorms of

* "Rainfall and Run-off in the Miami Valley," by Ivan E. Houck. Part VIII, Technical Reports of the Miami Conservancy District (1921), p. 20.

from 5 to 10 inches will ordinarily produce a run-off of between 50 per cent and 90 per cent.

These figures for the per cent of run-off are higher than usually considered as the value of C in the "Rational Method" used in sewer design, because while the value of C in the Rational Method is usually defined as being the per cent of run-off, actually it is the ratio between the rate of peak flow and the rate of the rainfall. Consequently the coefficient C includes a factor representing the other drainage area characteristics, such as pondage, and slope and distribution of drainage area which tend to reduce the peak flow below the rate of rainfall.

It is hoped that more information and data will be collected and published on this subject of the relations between flood run-off and rainfall. A heavy rain does not necessarily mean that serious flood conditions will result. However, with greater knowledge of rainfall-run-off relations it should be possible to forecast the probable flood stage with sufficient accuracy to warrant flood warnings much sooner than is possible when river stages alone are depended upon for information, particularly on small and flashy streams.

TABLE 14. — RAINFALL AND FLOOD RUN-OFF

*Miami River above Dayton, Ohio**

[Drainage area, 2,525 square miles]

DATE OF STORM	Average Rainfall above Station (Inches)	Flood Run-off (Inches)	Retention (Inches)	Run-off Percentage of Rainfall
March 23-27, 1913	9.60	8.76	.84	91.0
March 28-29, 1924	2.06	1.16	.90	56.0
June 8, 1924	2.22	1.73	.49	78.0
Sept. 12-13, 1925	3.20	.04	3.16	1.2
Nov. 12, 1925	1.43	.66	.77	46.0
Nov. 26-27, 1925	1.29	.20	1.09	15.6
April 7-8, 1926	1.23	.84	.39	68.0
Sept. 22-26, 1926	2.08	.39	1.69	18.7
Jan. 18, 1927	1.98	1.36	.62	68.7
March 19-21, 1927	3.44	2.31	1.13	67.0
May 18, 1927	2.05	.61	1.44	29.8
Nov. 30, 1927	2.15	.64	1.51	29.8
Dec. 13, 1927	1.24	.64	.60	51.6
April 21-22, 1928	2.08	.53	1.55	25.5
June 3-6, 1928	2.86	.46	2.40	16.1
Feb. 25-26, 1929	1.80	2.05	— .25†	114.0†

* Data from table prepared by C. S. Bennett, Engineer of the Miami Conservancy District, in Engineering News-Record, April 11, 1929, p. 600.

† Melting snow added to rainfall.

TABLE 14. — RAINFALL AND FLOOD RUN-OFF — *Continued**Murchison Farm Brook, Jackson, Tenn.**

[Drainage area, 112 acres]

DATE OF STORM	Average Rainfall above Station (Inches)	Flood Run-off (Inches)	Retention (Inches)	Run-off Percentage of Rainfall
Feb. 19, 1918	.95	.48	.47	50.5
April 16, 1918	1.55	.57	.98	36.8
April 19, 1918	1.08	.92	.16	85.2
May 7, 1918	1.31	.39	.92	29.8
May 12, 1918	.52	.18	.34	34.6
May 23, 1918	.92	.30	.62	32.6
June 1, 1918	1.14	.37	.77	32.5
June 6, 3 P.M., 1918	.33	.17	.16	51.5
June 6, 11 P.M., 1918	.71	.35	.36	49.3
July 18, 1918	2.17	.63	1.54	29.0

Nepaug River, Hartford, Conn., Water Works

[Drainage area, 23.9 square miles]

Nov. 3-4, 1927	5.88	3.04	2.84	51.7
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East Branch Farmington River

[Drainage area, 47.4 square miles]

Nov. 3-4, 1927	6.52	5.35	1.17	82.0
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Wachusett Watershed

[Drainage area, 109 square miles]

Aug. 1-6, 1915	4.97	1.52	3.45	30.6
June 22-23, 1918	2.73	.34	2.39	12.5
June 15-19, 1920	3.52	1.04	2.48	29.6
Sept. 28-30, 1920	4.54	.54	4.00	11.9
Nov. 21-24, 1920	2.69	.96	1.73	35.7
Dec. 13-14, 1920	2.12	.66	1.46	31.1
May 4-7, 1922	3.05	1.50	1.55	49.2
April 28-30, 1923	3.37	1.71	1.66	50.7
Nov. 2-4, 1927	4.90	2.23	2.67	45.5

* Data from "Run-off Small Agricultural Areas," by C. E. Ramser, in *Journal of Agricultural Research*, Vol. 34, No. 9, p. 806.

TABLE 14. — RAINFALL AND FLOOD RUN-OFF — *Continued**Ware River above Gibbs Crossing*

[Drainage area, 201 square miles]

DATE OF STORM	Average Rainfall above Station (Inches)	Flood Run-off (Inches)	Retention (Inches)	Run-off Percentage of Rainfall
April 29-30, 1921	2.72	1.17	1.55	43.0
June 15-18, 1920	3.35	.65	2.70	19.4
Sept. 28-30, 1920	4.33	.67	3.66	15.4
Nov. 21-24, 1920	1.75	.28	1.47	16.0
Dec. 13-14, 1920	1.64	.77	.87	46.9
May 4-7, 1922	2.45	.86	1.59	35.1
April 28-30, 1923	3.46	1.46	2.00	42.2
April 18-22, 1924	2.15	.98	1.17	45.6
Nov. 2-4, 1927	4.08	1.93	2.15	47.3

Swift River above West Ware

[Drainage area, 186 square miles]

June 15-18, 1920	2.90	.73	2.17	25.2
Sept. 28-30, 1920	3.71	.67	3.04	18.1
Nov. 21-24, 1920	1.59	.23	1.36	14.5
Dec. 13-14, 1920	1.76	.61	1.15	34.6
May 4-7, 1922	2.85	.71	2.14	24.9
April 28-30, 1923	3.18	1.30	1.88	41.0
April 18-22, 1924	2.49	.78	1.71	31.3
Nov. 2-4, 1927	4.36	1.72	2.64	39.5

Middle Branch of Westfield River above Goss Heights

[Drainage area, 53 square miles]

June 15-18, 1920	3.13	1.30	1.83	41.5
Sept. 28-30, 1920	4.20	.85	3.35	20.2
Nov. 21-24, 1920	2.62	1.37	1.25	52.3
Dec. 13-14, 1920	1.44	1.31	.13	91.0
May 4-7, 1922	2.26	1.26	1.00	55.7
April 28-30, 1923	1.78	.77	1.01	43.3
April 18-22, 1924	1.95	1.32	.63	67.7

TABLE 14. — RAINFALL AND FLOOD RUN-OFF — *Concluded**Westfield River above Westfield Gaging Station*

[Drainage area, 496 square miles]

DATE OF STORM	Average Rainfall above Station (Inches)	Flood Run-off (Inches)	Retention (Inches)	Run-off Percentage of Rainfall
Nov., 1919	2.78	.87	1.91	31.2
June 15-18, 1920	3.17	.84	2.33	26.5
Sept. 28-30, 1920	4.32	.84	3.48	19.5
Nov. 21-24, 1920	2.67	.94	1.76	35.2
Dec. 13-14, 1920	1.79	1.13	.66	63.3
May 4-7, 1922	2.27	1.10	1.17	48.5
April 28-30, 1923	1.88	.64	1.24	34.0
April 18-22, 1924	2.12	.87	1.25	41.0
Nov. 2-4, 1927	6.48	3.92	2.56	60.5

RUN-OFF FROM MELTING SNOW

There is but little information available upon the effect of snow accumulation upon flood flows in New England. Government rainfall, temperature and run-off records are valuable aids, but to date no snow accumulation records of value have been kept by government observers. Such snow records as exist are found to be those compiled by municipal water supply and power companies, the latter particularly finding such records of increasing value. Advance information permits the manipulation of storage systems to fit the power requirements, and the use of stored water to obtain the maximum good from the snow run-off.

While a heavy snow accumulation frequently causes anxiety along our rivers, particularly if it exists into the late spring, extreme floods are seldom caused from snow run-off alone. It requires very warm temperature of several days' continuous duration to cause a large flood from snow. Such run-off responds very quickly to temperature variations. A day or two of cooler temperature during several warm days has a very dampening effect on the run-off.

The outstanding example of a flood from snow alone with practically no rainfall occurred in April, 1862, at Brattleboro and Bellows Falls, on the Connecticut River. This flood was the record flood up to 1927, although several others have approached it in peak flow. At the

same time, there occurred a flood approaching the record on the Merrimack River at Lowell.

Another snow flood with practically no rainfall occurred on the Nepaug River, near Collinsville, Conn., in late March, 1916. This flood amounted to a peak flow of 27 second feet per square mile on a drainage area of about 27 square miles. However, in April, 1924, there occurred a peak flood of 90 second feet per square mile, due to excessive rain on this same stream.

Large floods on the Maine rivers occurred in late April and early May, 1923, which have been described as snow floods because of the large amount of snow on the ground at the time, but heavy rains occurred and the resultant flows were a combination of water from rainfall and snow. On the Penobscot the rainfall was 5.25 inches in three days, with a maximum 24-hour rainfall of 2.06 inches on the second day, and a maximum 24-hour run-off of 1.06 inches 2 days later. The maximum 3-day run-off totaled 2.99 inches, and the 10-day run-off totaled 6.76 inches, or 1.42 inches more than the total rainfall for the period. The Kennebec above Waterville had a maximum 24-hour flow of 1.38 inches, and a 10-day flow of 6.67 inches, and the Androscoggin, above Rumford, a maximum 24-hour flow of .80 inch and a 10-day flow of 4.95 inches.

Heavy rains with snow on the ground usually give a large volume of run-off extending over several days. Indications are, however, that the same rains occurring with little or no snow on the ground would produce greater peak flows than they do where there is sufficient snow to act like a sponge to flatten out the peaks. The study of such records as are available show that substantial rainfalls accompanied by warm temperature with snow on the ground produce run-offs over a period of days in excess of the total amount of the rain. This is to be expected. The vital question in any specific case is what the peak flow will be and when will it occur.

Such records as are at hand have been studied hoping to find some way to solve this question for a given set of conditions. It is felt that some progress has been made along this line, and it is hoped that the preliminary results here given may encourage others to continue the study along similar or other lines, or, at least, that the information will demonstrate the desirability of obtaining more records of snow accumulation and accompanying conditions affecting snow run-off.

It is recognized that a number of factors enter into the problem of run-off from snow, such as sunshine, rain, wind, temperature and ground conditions, but studies so far indicate that air temperature is the pre-

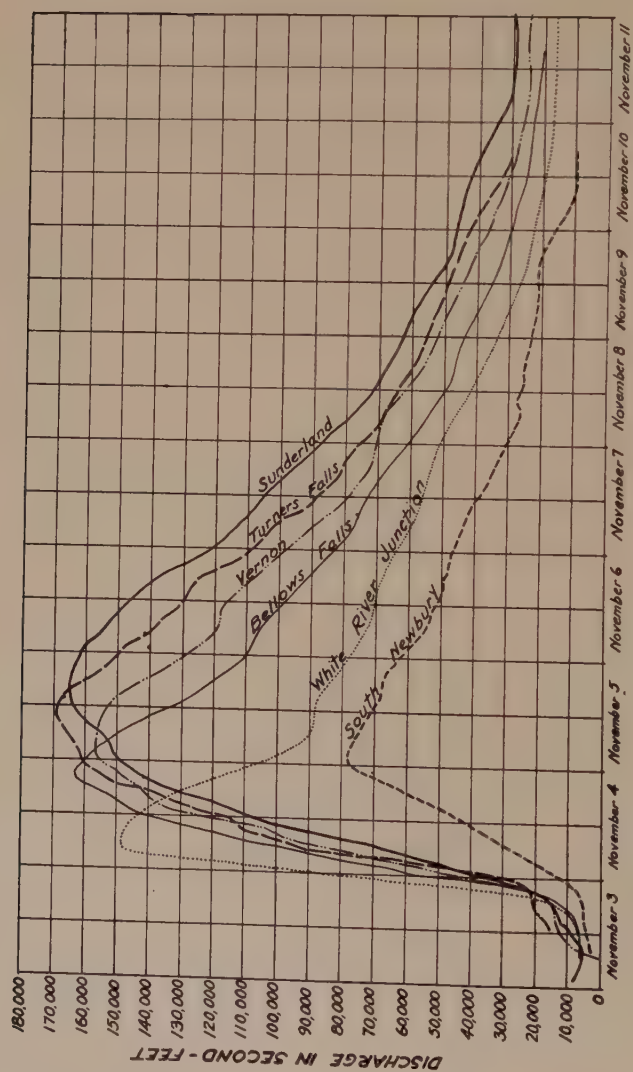


FIG. 3. — HYDROGRAPHS OF DISCHARGE AT POINTS ON THE CONNECTICUT RIVER DURING FLOOD OF NOVEMBER, 1927

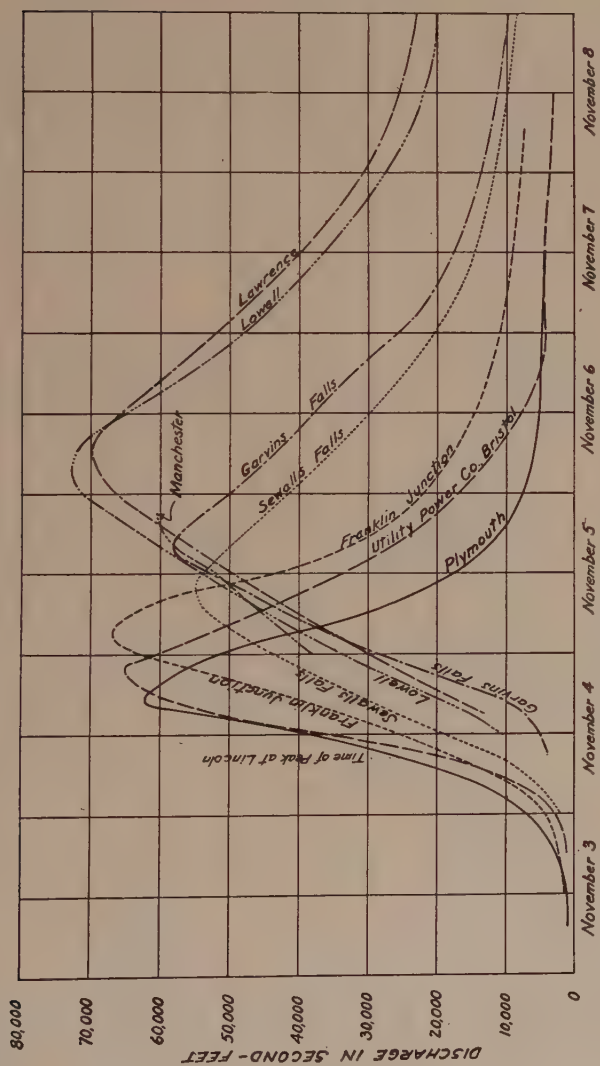


FIG. 4. — HYDROGRAPHS OF DISCHARGE AT POINTS ON THE MERRIMACK RIVER DURING FLOOD OF NOVEMBER, 1927

dominating influence, and rainfall appears to have a very definite effect, as shown later.

To study the effect of temperature on snow run-off, the unit of "degree day" has been used. This unit is a degree of temperature for one day of time, and the attempt has been to find a value of run-off from snow for each degree day of temperature above some given base. The temperature used is the 24-hour mean as derived from the maximum and minimum readings.

A mathematical analysis of the data resulted in the following formula:

$$R = .738 + .504 P + .0126 (T - 27.1) D$$

R = Run-off in inches

P = Precipitation during the period in inches

T = Average temperature for the period in degrees Fahrenheit

D = Number of days in the period

$(T - 27.1) D$ = Degree days

The base temperature of 27.1° in this formula appears low, but considering that a mean temperature above 27 usually involves a temperature during part of the 24 hours above the freezing point, it seems reasonable that the base temperature should be below 32° . That half of the precipitation should appear as run-off during a snow run-off period also seems reasonable.

In the derivation of this formula, no allowance was made for the ground water flow. It is probable that the constant .738 represents an average value for this ground water flow during the periods considered, although it may include other factors. It is equivalent to about 2 second feet per square mile for a 10-day period.

To determine a coefficient of run-off per degree day the equation was rewritten as follows:

$$R = .5 P + c_s (T - 27) D, \text{ or, } c_s = \frac{R - .5 P}{(T - 27) D}$$

By using this equation the coefficient c_s has been obtained for 34 periods covering data on several rivers, and the results are given in Table 15. The table also shows total run-off for the given periods, and the run-off for the maximum 24 hours. These figures give a good idea of snow run-off totals for these several streams.

The run-off per degree-day is also shown graphically on Fig. 5. On this diagram is plotted the total run-off less half of the precipitation against total degree days above 27° for the period. The snow run-off

rate for any point on the diagram can be read directly by drawing a line from the origin to the point and reading where the line cuts the vertical 100 degree day line. For one degree day the decimal point in the run-off scale is moved two points to the left. From this diagram it will be observed that except for three periods on the Deerfield River all values lie between .010 and .030, the mean being about .020. The most consistent group of observations is that covering the Androscoggin River. The Deerfield River observations run considerably higher than the three Maine rivers, while all snow floods of the Merrimack River in 1862, the Nepaug River in 1916, and the maximum day flood of the Connecticut River for 1862, all lie a little below the mean line. It is

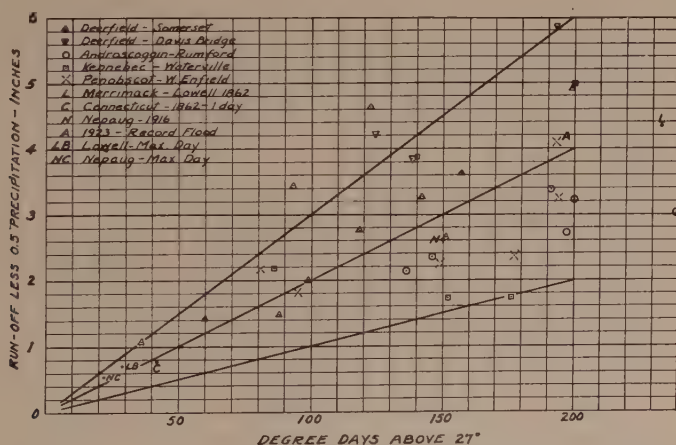


FIG. 5. — RELATION BETWEEN RUN-OFF FROM MELTING SNOW AND TEMPERATURE

also interesting to observe on the diagram that the maximum day relations for the Merrimack, 1862, and the Nepaug, 1916, points, L. B. and N. C., are only slightly higher than the total period values. This maximum day relation is expressed in terms of the maximum mean daily temperature as reflected in the maximum daily run-off of 24 or 48 hours later. It is this maximum day relation which may make this whole snow run-off study of value in predicting a day or two in advance the probable maximum flood for a given stream.

The "A" on the diagram shows the results of the study of the 1923 record flood on the Penobscot River. It is interesting that the period relation is very consistent with other results, even with a period rainfall of 5.34 inches. In this case the maximum day temperature run-off rela-

tion does not hold true, but it may be significant that the maximum daily run-off was half of the maximum 24-hour precipitation, and occurred 48 hours later. Had there been less snow on the ground, might not the peak have been in excess of 1.06 in 24 hours?

Conclusions

Record floods in New England, due to the effect of snow run-off, are rare, but may occur any time a heavy snow accumulation is present.

Excessive rainfall with complete snow cover with the accompanying high temperature may result in a peak 24-hour run-off equal to half of the maximum 24-hour rainfall.

With a less complete snow cover the same rainfall may cause an even higher peak flow.

Approximate peak run-off under snow-covered conditions, without rain, can be predicted a day or two in advance by using the value of .020 to .025 inches per degree day excess above 27° based on the mean 24-hour temperature.

For smaller drainage areas at higher elevations, the degree day run-off relation appears to run higher than .025.

For moderate amounts of rainfall, say, up to 1 inch, probably the run-off degree day relation also holds for peak day run-off if half of the precipitation is deducted before applying the degree day coefficient.

TABLE 15. — SNOW RUN-OFF DATA

Deerfield River at Somerset, Vt.

[Drainage area, 30 square miles]

PERIOD	Days	Precipitation for Period	Natural Run-off	Snow Run-off (Total Run-off Less Half of Precipitation)	Maximum Run-off in 24 Hours	Degree Days above 27°	Snow Run-off per Degree Day above 27°
April 17-30, 1917	14	1.43	5.64	4.92	1.00	199	.025
April 21-30, 1918	10	1.36	3.34	2.66	.43	151	.018
March 24-28, 1921	5	1.31	2.12	1.50	.64	88	.017
April 6-11, 1922	6	.60	3.10	2.80	2.17	118	.024
April 21-28, 1923	8	.30	3.78	3.63	.93	157	.023
April 27-30, 1924	4	.00	1.44	1.44	1.15	60	.024
March 26-28, 1925	3	.53	1.35	1.07	1.53	36	.030
May 1-8, 1926	8	.25	4.76	4.64	.99	122	.038
March 12-19, 1927	8	.00	3.45	3.45	.78	93	.037
April 16-22, 1927	7	.00	3.28	3.28	.62	142	.023
April 3-7, 1928	5	.00	2.01	2.01	1.11	99	.020

TABLE 15. — SNOW RUN-OFF DATA — *Continued**Deerfield River at Davis Bridge, Vt.*

[Drainage area, 154 square miles net]

PERIOD	Days	Precipitation for Period	Natural Run-off	Snow Run-off (Total Run-off Less Half of Precipitation)	Maximum Run-off in 24 Hours	Degree Days above 27°	Snow Run-off per Degree Day above 27°
April 21–30, 1926 .	10	.92	6.32	5.86	1.13	193	.031
March 12–19, 1927 .	8	.09	4.26	4.22	.88	124	.034
April 3–8, 1928 .	6	.19	3.95	3.85	1.05	138	.028

Merrimack River at Lowell, Mass.

[Drainage area, 3,570 square miles net]

April 15–27, 1862 .	13	.90	4.79	4.34	.70	233	.019
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Connecticut River at Vernon, Vt.

[Drainage area, 6,300 square miles]

April 20, 1862 .	1	.00	.79	.79	.79	42*	.019
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* From Lowell record plus difference between reported maximums at Lowell and Brattleboro, Vt.

Nepaug River, near Collinsville, Conn.

[Drainage area, 27 square miles]

March 26–April 3, 1916 . . .	9	.00	2.62	2.62	.55	150	.017
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Androscoggin River at Rumford, Me.

[Drainage area, 2,090 square miles]

March 27–April 5, 1921 . . .	10	1.18	2.74	2.15	.36	136	.016
April 9–18, 1922 .	10	1.66	4.22	3.39	.81	191	.018
April 28–May 7, 1923	10	3.86	4.95	3.02	.80	239	.013
May 1–10, 1924 .	10	1.56	3.50	2.72	.46	197	.014
March 28–April 6, 1925 . . .	10	1.51	3.13	2.37	.51	146	.016
May 2–11, 1926 .	10	.36	3.40	3.22	.54	200	.016

TABLE 15. — SNOW RUN-OFF DATA — *Concluded**Kennebec River at Waterville, Me.*

[Drainage area, 4,270 square miles]

PERIOD	Days	Precipitation for Period	Natural Run-off	Snow Run-off (Total Run-off Less Half of Pre- cipitation)	Maximum Run-off in 24 Hours	Degree Days above 27°	Snow Run-off per Degree Day above 27°
March 25–April 3, 1921	10	1.45	2.91	2.19	.39	86	.025
April 12–21, 1922 .	10	.77	4.26	3.88	.62	140	.028
April 29–May 8, 1923	10	3.37	6.67	4.98	1.38	200	.025
April 30–May 9, 1924	10	1.69	2.58	1.74	.42	152	.011
May 1–10, 1926 . .	10	.62	2.05	1.74	.34	176	.010

Penobscot River at West Enfield, Me.

[Drainage area, 6,600 square miles]

March 26–April 4, 1921	10	1.02	2.70	2.19	.32	81	.027
April 11–20, 1922 .	10	1.42	2.96	2.25	.50	149	.015
April 28–May 7, 1923	10	5.34	6.76	4.09	1.06	193	.021
April 29–May 8, 1924	10	1.42	3.08	2.37	.38	177	.013
March 30–April 8, 1925	10	.65	2.15	1.83	.37	95	.019
May 2–11, 1926 . .	10	.84	3.67	3.25	.44	194	.017

DRAINAGE AREA CHARACTERISTICS

The importance of the differences in drainage area characteristics has been generally recognized, but not usually sufficiently emphasized. In many of our New England streams, they are of much greater significance in their effect on floods than the size of the drainage area.

For the purpose of simplifying the discussion which follows, it is assumed that the run-off equals the rainfall. Usually this is not the case, but with certain conditions of the ground, such as when frozen or when already saturated by previous rains, the run-off can closely approach the rainfall, and with snow on the ground and high temperatures, might conceivably exceed it. It will further clarify the discussion to express both rainfall and run-off as inches depth on the drainage area.

Assuming a rainfall at a uniform rate for a sufficient length of

time, and neglecting infiltration, the unit run-off at any point on most drainage areas will eventually equal the rate of rainfall. Under such conditions the flow at any point will vary directly with the rate of rainfall and with the size of the drainage area. Under this conception, all drainage areas would behave substantially the same, and the differences in special drainage area characteristics, while affecting the length of time for the peak to be reached, would not affect the magnitude of the peak, expressed in terms of unit run-off. The rate of rainfall multiplied by the drainage area would be the universal flood formula.

Actually, however, this assumed condition of a storm continuing until the run-off equals the rainfall exists only on the smallest drainage areas, and on these the size of floods will entirely depend upon the rainfall rates over relatively short periods of time.

But as the length of a stream or the amount of pondage or storage increases, the time required for the run-off water to flow from its sources to its mouth also increases, and there must be an increase in the length of time that a storm must be maintained in order to produce a stable condition, with the unit run-off equaling the rate of rainfall throughout the drainage area.

The climatic conditions in the northeastern part of the United States are such that, as the duration of the storm increases, the average intensity decreases with marked rapidity. The rainfall to be expected with a 50-year frequency for various periods of time from the Chestnut Hill record (see Table 9) is repeated in Table 16, with the corresponding average intensities in inches per hour and in second feet per square mile.

TABLE 16. — RAINFALL AT CHESTNUT HILL, 50-YEAR FREQUENCY

LENGTH OF STORM (HOURS)	Total Rainfall	Average Intensity in Inches per Hour	Average Intensity in Second Feet per Square Mile
1	2.42	2.42	1,560
2	3.10	1.55	1,000
3	3.55	1.18	762
5	4.18	.84	541
8	4.84	.60	387
12	5.52	.46	296
24	6.80	.28	180
36	7.68	.22	142
48	8.37	.17	110
60	8.95	.15	97
72	9.45	.13	84
96	10.30	.11	71

The table shows the importance of the time required for the run-off to equal the rainfall. For example, the probable intensity of a 2-hour storm is at the rate of 1,000 second feet per square mile, while the probable 48-hour storm is at the rate of 110 second feet per square mile.

Thus the magnitude of the unit run-off in times of flood will depend upon the *rapidity of the run-off*, which is determined primarily by three features of the drainage area above any point:

1. The magnitude and distribution of the distances and areas above it.

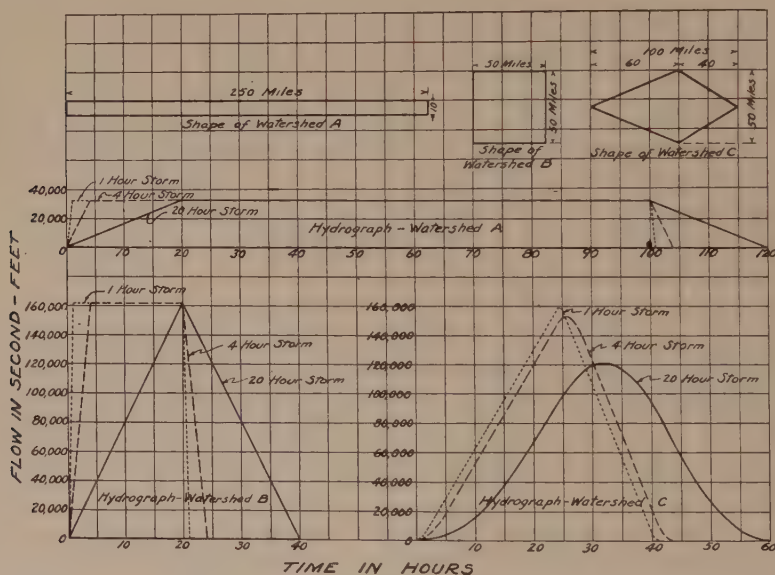


FIG. 6. — DIAGRAM SHOWING EFFECT OF SHAPE OF DRAINAGE AREA

2. The velocity of flow which determines the rapidity with which the run-off reaches it.

3. The amount of pondage or storage available to absorb and retard the natural run-off.

Of these, the first, *i.e.*, the magnitude and distribution of the distances and areas, may be considered the fundamental feature, with the last two exerting important modifying influences.

The Influence of Size and Distribution of Drainage Areas upon Flood Flows

In order to demonstrate the influence of the shape of a drainage area upon flood flows, the theoretical flow has been computed for three differently shaped areas, each having a total area of 2,500 square miles. (See Fig. 6.) Watershed "A" is 250 miles long and 10 miles wide; watershed "B" is 50 miles long and 50 miles wide; while watershed "C" is assumed to have a diamond shape with a length of 100 miles and a maximum width of 50 miles. It is assumed that the flood water will flow with a uniform velocity of $2\frac{1}{2}$ miles per hour along the thread of the stream for each of these. The time required for flow from the outer edges of the drainage area to the thread of the stream and pondage have been neglected.

Three storms, each with a total rainfall and run-off of 2 inches from the entire drainage area, have been assumed, as follows:

- (a) Two inches per hour for 1 hour.
- (b) One-half inch per hour for 4 hours.
- (c) One-tenth inch per hour for 20 hours.

The resultant flow at the lower end of each of these watersheds for these storms, is shown in Fig. 6. It will be noted that all three storms produce a peak flow of 32,250 cubic feet per second on watershed "A," although in each case this peak is not reached until the end of the storm. On watershed "B" the peak flow becomes 161,500 second feet, or five times the peak for watershed "A," which is in proportion to the widths of the two drainage basins. As in the case of watershed "A," the peak flow is reached at the end of the storm. On watershed "C" the results for the three assumed storms are no longer the same. The peaks and their magnitudes are reached as follows:

One-hour storm, 158,000 second feet in $24\frac{1}{2}$ hours.

Four-hour storm, 153,000 second feet in 26 hours.

Twenty-four-hour storm, 121,000 second feet in 32 hours.

Watershed "C" has the same maximum width as watershed "B," and the peak flow for the 1-hour storm on "C" is nearly as large as that for "B." If the 2 inches of precipitation had been assumed to fall instantaneously on "C," the peak would have been 161,000 second feet, or the same as for "B." Thus the size of the peak flow for "C" depends upon the maximum average width of the watershed over such a distance as is traversed by the flow during the period of the storm. Had the storm occurred at the rate of .05 inch per hour for 40 hours, which is the time required for the run-off to flow from the source to the mouth,

the peak flow would have been one-half of 161,000 second feet, or 80,500 second feet.

Summarizing the conclusions to be drawn from these three simple drainage areas, it appears that for an instantaneous storm the peak flow will vary directly with the maximum width. For storms of longer duration the flow will depend upon the greatest average width of the drainage basin for such a distance as is covered by the flow of the water during the assumed period of the storm.

If the three watersheds had been similar in shape, it would follow from geometrical relations that the corresponding widths would vary

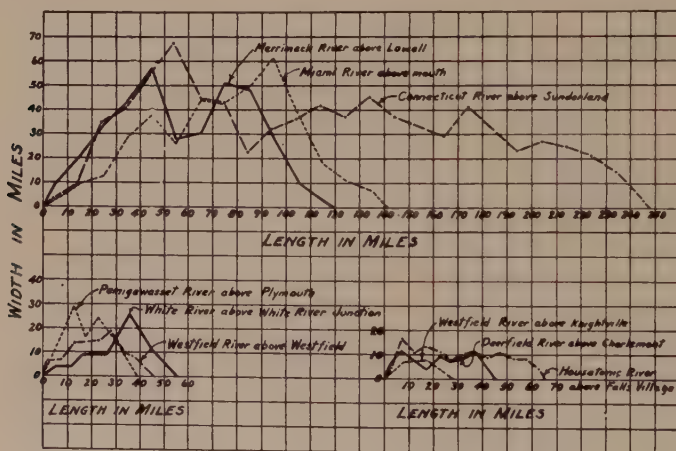


FIG. 7. — EFFECTIVE WIDTHS OF SEVERAL NEW ENGLAND DRAINAGE AREAS

directly with the square roots of the drainage areas. Hence, the conclusion can be drawn that for identical storms (within the concentration period) on similarly shaped watersheds having the same velocities and negligible pondage, the peak flows will vary directly with the square root of the drainage areas. The peak unit flow will then vary inversely with the square root of the drainage area.

The widths of drainage areas for several New England rivers at given distances above specified points are shown on Fig. 7. They have been determined by starting at the point in question and drawing distance lines at 5 and 10 mile intervals. The areas between these distance lines were divided by the distance intervals, giving what is called the effective width of the drainage area.

The Effect of Velocity of Flow on Flood Peaks

For identical storms on each of two similar drainage areas, the maximum discharge will vary directly with the velocity of flow in the two streams, other conditions being the same. As every river is made up of a number of tributaries, all of which have varying degrees of slope, generally increasing as the headwaters are approached, it is difficult to give any single figure as representing the velocity of an entire river.

Streams having steep slopes are subjected to sudden and heavy

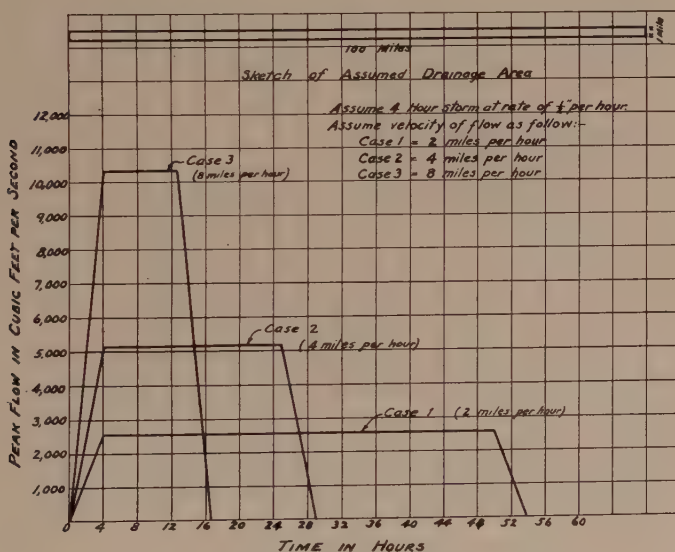


FIG. 8. — DIAGRAM SHOWING EFFECT OF VELOCITY

freshets, while streams having flat and gentle slopes show much more even flow, since the steep drainage area pours down its flood run-off in a short period of time, while the flatter drainage area spreads a total flood of equal volume over a much longer interval with a correspondingly lower peak.

A consideration of the behavior of three imaginary drainage areas, each 100 miles long and 1 mile wide, as shown in Fig. 8, will show the direct relation between the velocity of flow and the volume of the peak. The slope of each is assumed to be such that the run-off flows with a uniform velocity from the source to the mouth, and the effect

of pondage has been neglected. It is further assumed that on drainage area "A" the water flows with a uniform velocity of 2 miles per hour, on "B" at 4 miles per hour, and on "C" at 8 miles per hour, also that all three of these drainage areas are subjected to a storm of 4 hours' duration, at the rate of one-half inch per hour, and with 100 per cent run-off. During these 4 hours the water which first falls on drainage area "A" will have traveled a total distance of 8 miles. Hence, the flow at the mouth will have steadily increased at the rate of 322.5 second feet as each additional square mile of the drainage area adds its flow. At the end of the 4 hours the water from the upper end of the 8 miles will have reached the mouth, and the total flow will have become 8×322.5 , or 2,580 cubic feet per second. This flow will be maintained steadily for a period of 46 hours more, or until 50 hours from the beginning of the storm, and will then drop off during the next 4 hours at a uniform rate to zero flow.

The behavior of drainage area "B" would be similar, except that during the 4 hours of the storm the water would have traveled a distance of 16 miles, and the peak flow would become 5,160 second feet. This peak flow would be maintained for 21 hours, which would be 25 hours from the time of the beginning of the storm, and then would fall off to zero during the next 4 hours.

The flow in the case of drainage area "C" would be still further concentrated, as with water traveling at the rate of 8 miles per hour, a distance of 32 miles would be covered during the period of the storm, and the peak flow would be 32×322.5 , or 10,320 second feet. In this case it would be maintained only $8\frac{1}{2}$ hours, and again would drop to zero during the succeeding four hours.

This simple diagram illustrates the manner in which a quick run-off is concentrated to produce high peak flows, in contrast to a slow run-off which is spread out over a much longer period of time, so that for similar drainage areas, and with all other factors being equal, the peak of flood flows will vary directly with the velocity of flow.

Flood slopes would appear to give the best measure of the velocity of flow which is available. In the Chezy formula, $V = C\sqrt{RS}$, if differences in C and R are neglected the velocity will vary as the square root of the slope. There are, of course, substantial differences in the hydraulic radii of different streams. If 5 feet is taken as representing the hydraulic radius of the smaller streams under flood conditions, and 25 that of the larger streams of New England under flood conditions, the differences in velocity of flow due to the difference in the hydraulic radii will be as the square roots of these figures, or as 2.24 is to 5. This

wide difference in the hydraulic radius makes a difference of 100 per cent in the velocity.

Considering the flood slopes in feet per mile, these will vary from about 1 foot per mile for the large streams to 50 feet per mile for steep rivers such as the Deerfield. With the velocity varying as the square root of the slopes, this difference makes a variation between 1 and 7.07, or 700 per cent.

Generally speaking, the streams with the flatter flood slopes are likely to have larger hydraulic radii than those with steep slopes, although this is not necessarily the case. The effect of the larger hydraulic radii of the streams of flatter slopes is to increase the velocity for a given slope, so that, to a certain extent, the effect of the differences in the hydraulic radii is to offset in part the differences in flood slopes.

In considering the flood slope of any section of a river, any abrupt fall at dams and rapids which still remains under flood conditions must be deducted. Such falls located between flat stretches of river have but little effect on the velocity of flow of the river considered as a whole. The small cascades which form on most small brooks and streams having steep slopes frequently use up a large part of the fall, so that the actual velocity of the water is usually less than on the larger streams where the slopes are much flatter, but where the body of water is sufficient to give a deeper cross section that to a large degree eliminates the effect of the irregularities in the bed of the stream. This effect makes the situation too involved in such cases to arrive at any definite conclusions regarding the relations between flood slopes and velocities.

The changes in slope and velocity in a given stream for various stages of flow must also be considered. If river channels had vertical sides or sloping banks of a uniform nature, then with a rising stage the flood slope and the hydraulic radius would both increase, with a resulting increase in velocity. Under these conditions the greater the storm the higher the velocity of flow, and the shorter the time of concentration. In an extreme case, if a 2-inch storm had a concentration period* of forty hours the flood due to a 4-inch storm might travel with a sufficiently high velocity to have a concentration period of 20 hours. Doubling the storm would not only double the volume of water flowing, but would also cut in half the time over which the run-off would be distributed, and hence would quadruple the peak flow.

In most cases, however, rivers overflow their banks under extreme flood conditions. The width of the streams tends to increase much more rapidly than the depth. In fact, due to overflowing the lowlands

* See p. 265.

on both sides of the stream, the mean hydraulic radius for the entire cross section of a river may be actually reduced with a rising stage. Furthermore, the frictional resistance, due to trees, bushes and other obstructions, increases.

An indication of the changes in the mean velocity of flow in streams under flood conditions is given by the velocity curves at the stream gaging stations of the United States Geological Survey. The velocity in practically all cases increases at a much slower rate as the river stage

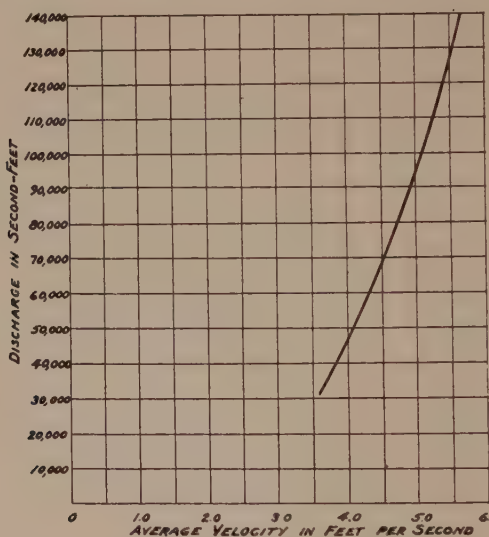


FIG. 9. — FLOW v . MEAN VELOCITY FOR CONNECTICUT RIVER, SUNDERLAND, MASS.

rises. In fact, in many cases the velocity appears to approach a fixed quantity. The result of a typical velocity curve is given in Fig. 9.

On the other hand, in steep streams confined to a narrow valley, the velocity does continue to increase steadily with a rising stage. The average velocities as given in Table 17 were computed by the United States Geological Survey for the flood of 1927. These velocities are doubtless much greater than those occurring in floods of half the magnitude of 1927. This is indicated by the large sizes of stones and boulders moved by the 1927 flood as compared with those moved by previous floods of lesser magnitudes. It will be noted that in all cases these high velocities were obtained on streams having steep slopes, and, for the most part, relatively narrow flood valleys.

TABLE 17. — AVERAGE VELOCITIES FOR 1927 FLOOD

RIVER AND STATION	Drainage Area in Square Miles	Discharge in Second Feet	Discharge in Second Feet per Square Mile	Area of Cross Section in Square Feet	Hydraulic Radius in Feet	Velocity in Feet per Second
Androscoggin River Basin:						
Peabody River, at Glen House	17.4	7,330	421	572	4.78	12.8
Peabody River, at Gorham	40.0	9,920	248	1,040	7.46	9.5
Saco River Basin:						
Ellis River, at Jackson	28.0	14,800	528	884	6.56	16.8
Merrimack River Basin:						
Mad River, above Campton Village	47.0	10,200	217	1,030	6.27	9.9
Bakers River, at Wentworth	52.0	15,000	288	1,130	11.10	13.3
Connecticut River Basin:						
Mohawk River, Colebrook, first location	26.0	4,340	167	426	3.98	10.2
White River, above White River Junction	695.0	140,000	202	8,690	21.70	16.1
Cold River, Charlemont	32.2	7,760	241	712	5.82	10.9
St. Lawrence River Basin:						
Winooski River, Montpelier	397.0	60,600	153	7,870	11.60	7.7

The Effect of Channel Pondage in Reducing Flood Peaks

Channel pondage, meaning the pondage obtained on a river in the space occupied by the rise of water from normal to flood stage, has long been recognized as a factor in the maximum flood flow at many points. If a flood flowing into a pond can be represented by a triangular hydrograph, such as AB and BC in Fig. 10, and the outflow can be represented by lines AD and DE, the effect of the pondage will be to reduce the peak flow in the direct proportion that the volume of the pondage bears to the total volume of the flood run-off.

The vertical ordinate between the lines AB and AD, such as MN, represents the excess of the inflow over the outflow during the period that the water is being ponded. Furthermore, the area of the triangle AMN from A to any ordinate, such as MN, represents the volume of the pondage up to the time at which the intersecting ordinate is drawn.

Under these assumptions the area of triangle ABC equals the area of triangle ADE, and equals the total volume of flood run-off *R*, also,

the area of triangle ABD represents the maximum volume of pondage P . Let y equal the reduction in peak flow.

$$R = \frac{b h}{2} \quad (1)$$

$$\text{and} \quad P = \frac{x h}{2} \quad (2)$$

$$\text{and} \quad \frac{x}{b} = \frac{y}{h} \quad (2a)$$

$$\text{Hence} \quad y = \frac{P}{R} h \quad (3)$$

Or the reduction (y) in the peak flow (h) due to the pondage (P) is directly proportional to the ratio of the volume of pondage (P) to the total flood run-off (R).

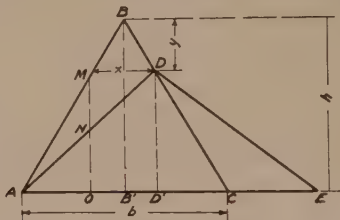


FIG. 10. — DIAGRAM SHOWING
EFFECT OF PONDAGE

The correctness of the assumption of a triangular flood hydrograph is indicated in the discussion of the effect of distribution of area and velocity on flood run-off. The hydrographs of the flood flow on the large majority of streams are similar to those shown in Fig. 10, although more or less curved.

As to the assumption of a straight line outflow, this will depend upon the nature of the discharge channel, and also upon the shape of the pondage basin. NO in Fig. 10, which represents the outflow from the pondage basin at any time, varies with the square root of the area of the triangle AMN, which area represents the volume of pondage accumulated to that time. This relation holds good from A to B. After passing the line BB', or the point of peak inflow, the volume of storage continues to increase, but at a rapidly decreasing rate, while the assumed rate of outflow continues to increase at the same rate. Hence, the relation as shown assumes that the outflow varies as the square root of the pondage from the point of time A to B, but that from B to D the outflow continues to increase at the same rate, which is greater than the square root of the increase in the pondage.

How do these two assumptions compare with normal river conditions in which the flow at any point is controlled either by a dam or by what is known as channel control, which is a combination of river slope and cross-sectional area? In either of these cases it is reasonably accurate to assume that the discharge varies as the $3/2$ power of the depth. Assuming that the control for the pondage under discussion is at a dam, the discharge will be —

$$Q = C_1 H^{3/2} \quad (4)$$

where Q = the discharge in cubic feet per second

C_1 = a constant, including both the length of the dam and the customary coefficient for the dam formula

and H = the depth on the dam or control in feet

The relation between the volume of pondage and the depth on the dam will vary with the shape of the pondage basin. Using the relation found by the Miami Conservancy District for the volume and depth of their storage reservoirs, viz.:

$$P = C_2 H^{5/2} \quad (5)$$

where P represents the volume of pondage in cubic feet

C_2 = a constant

H = the depth of pondage in feet

Solving equations (4) and (5) for Q gives —

$$Q = \frac{C_1}{C_2^{.6}} \times P^{.6} \quad (6)$$

Thus under these assumptions the discharge will vary as the 0.6 power of the pondage instead of the 0.5 power as was assumed for the greater part of the ponding period in Fig. 10.

During the latter part of the ponding from BB' to DD' in Fig. 10, the outflow varies at a higher rate than the 0.5 power of the pondage. The nature of all these assumptions is so general that it hardly seems necessary to determine whether the effect of period BB' to DD' is such as to make the relation between the outflow and the pondage approximate the relation Q varies as $P^{0.5}$, for the ponding period as a whole. This mathematical analysis indicates that there is at least a rough agreement between the assumption made and the conditions as actually found. Under this condition of an overflow dam and a storage basin it is believed that the straight line assumption for the outflow is a fair approximation, and that the general conclusion, as given at the start, holds good.

An estimate has been made of the volume of channel pondage on

the Connecticut and Merrimack Rivers during the flood of 1927. The results are shown in Table 18. It will be observed that on both of these rivers the channel pondage amounted to between .5 and 1.2 inches on the uncontrolled drainage area.

The effect of this channel pondage is shown in Figs. 3 and 4, giving the hydrographs of the flow at various points on these two rivers. The changes in the shape of the hydrographs on the Connecticut River between White River Junction and points farther down, and on the Merrimack River below Franklin Junction, show the influence of the channel pondage.

TABLE 18. — ESTIMATED CHANNEL PONDAGE
Connecticut and Merrimack Rivers — 1927 Flood

STATION	DRAINAGE AREA		VOLUME OF CHANNEL PONDAGE		
	Con- trolled (Square Miles)	Uncon- trolled (Square Miles)	Between Stations, M. C. F.	Total M. C. F.	Total in Inches on Uncon- trolled Drainage Area
<i>Connecticut River</i>					
White River Junction	4,120	4,040	6,450		
Bellows Falls	5,450	5,370	3,750	6,450	.52
Vernon	6,300	6,220	2,510	10,200	.71
Turners Falls	7,250	7,170	3,360	12,710	.76
Sunderland	8,000	7,740	6,180	16,070	.89
Holyoke	8,390	8,120		22,250	1.18
<i>Merrimack River</i>					
Plymouth	615	599	500		
Bristol	760	686	650	500	.31
Franklin Junction	1,460	935	1,630	1,150	.53
Sewalls Falls	2,280	1,755	1,400	2,780	.68
Garvins Falls	2,340	1,815	630	4,180	.99
Manchester	2,840	2,320	2,280	4,810	.89
Lowell	4,097	3,452	360	7,090	.89
Lawrence	4,663	3,930		7,450	.82

On the Connecticut River the peak flow at Bellows Falls was but $9\frac{1}{2}$ per cent higher than the peak at White River Junction, in spite of the fact that the drainage area is 32 per cent greater at Bellows Falls. The distinctly different shapes of the flood hydrographs at these two points shows the effect of the pondage in between them.

The situation is also illustrated by Fig. 11, which shows the effect of the pondage between Franklin Junction and Garvins Falls on the Merrimack River. The flow at Franklin Junction plus the estimated inflow between Franklin Junction and Garvins Falls gives a peak flow

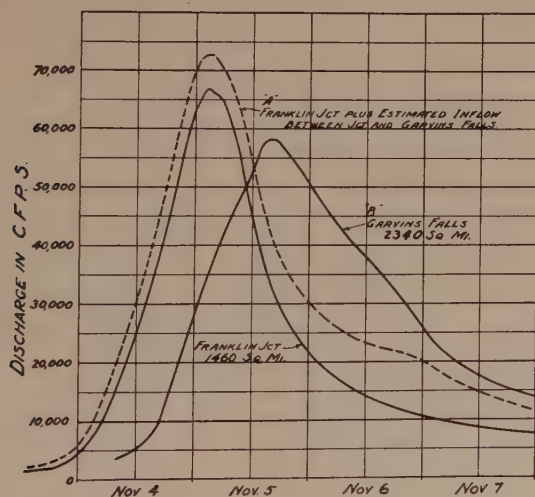


FIG. 11.—HYDROGRAPH SHOWING EFFECT OF PONDAGE ON MERRIMACK RIVER BETWEEN FRANKLIN JUNCTION AND GARVINS FALLS

of 73,520 second feet. The maximum flow over the Garvins Falls dam was estimated at 58,200 second feet. This represents a reduction of 20.8 per cent in the peak caused by 3,000,000,000 cubic feet of natural pondage, which is equivalent to about 29 per cent of the total flood volume at Garvins Falls in 1927.

An unusual case of channel pondage also occurs upon Otter Creek above Middlebury in Vermont. The dam at Middlebury is at about El. 336, while for many miles upstream the valley is wide and flat so that it overflows in high water every year. The water level reached in the 1927 flood above Middlebury was about El. 352, corresponding to a pondage of about 4.5 billion cubic feet or 7.0 million cubic feet per

square mile of drainage area. As a result in this flood the maximum flow at and below Middlebury was only about 23 second feet per square mile compared with 100 or more on many other rivers of this size at that time.

A similar effect of channel pondage is found on the Miami River in the 1913 flood.* The total flood run-off at Hamilton was estimated at 8.21 inches, with an estimated channel storage of 2.9 inches on the drainage area, or 35.4 per cent of the run-off. The actual peak at Hamilton was recorded as 350,000 c.f.p.s., and the estimated peak, as it would have been without this pondage, at 500,000 c.f.p.s. This difference means a reduction of 30 per cent in the peak.

Conclusions

From the preceding consideration of drainage area characteristics the following conclusions are drawn:

1. *Size and Distribution.* — For an instantaneous storm the peak flow will vary directly with the maximum width of the drainage area, all other conditions remaining the same. For storms of longer duration the peak flow will vary directly with the average maximum width of the drainage area for such a distance as is traveled by the flow during the period of the storm, all other conditions remaining the same. For similarly shaped drainage areas the peak flows tend to vary with the square root of the drainage areas.

2. *Velocity.* — Unit peak flows will vary directly with the velocity of flow, all other conditions remaining the same.

3. *Pondage.* — Pondage tends to reduce the peak flow in the direct ratio that the volume of pondage bears to the total flood run-off.

Chapter V. — Flood Formulæ for New England

FORMULA BASED ON FLOOD HYDROGRAPH

No two streams have the same shape, slope and distribution of tributaries. There are an infinite number of combinations of the relationships resulting from the shape and arrangement of different drainage areas. Furthermore, the amount of storage and pondage which comes into play under flood conditions not only varies widely for different streams, but also changes for floods of varying magnitude on the same

* Technical Reports of the Miami Conservancy District, Vol. 7, pp. 316-318; Vol. 8, p. 189.

stream. In very few cases has a quantitative determination of the amount of channel pondage been attempted, and even to estimate the amount held in the small depressions in the fields and valleys which go to make up a drainage area is an impossibility. The problem is extremely complex when approached analytically from its component parts of distances, slopes and pondage.

While it is not possible to determine these relationships definitely, all these features are automatically reflected in the flood hydrograph. It is believed that the flood hydrograph resulting from a given storm on a stream is the best key to the behavior of that stream with other storms, and also under different conditions of storage or pondage.

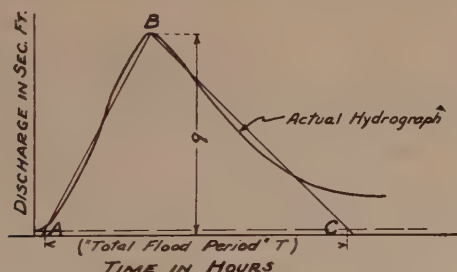


FIG. 12. — DIAGRAM SHOWING METHOD OF OBTAINING TOTAL FLOOD PERIOD FROM FLOOD HYDROGRAPH

The relationship between the peak flow and the time of run-off can be expressed mathematically. Referring to Fig. 12, the curved line is assumed to represent the actual flood hydrograph at a given point, with the flow in second feet per square mile plotted against time in hours. If two straight lines are drawn forming a triangle ABC, such that the area of the triangle is equal to the area under the flood hydrograph, then the area under each of these curves will equal the total volume of run-off.

Let R = the flood run-off in inches on the drainage area,
 q = the peak flow in second feet per square mile,
 and T = what will be called "total flood period in hours."

This "total flood period" is to be taken as the time in hours forming the base of the triangle ABC. It should be noted that T is determined by the straight lines which balance the flood hydrograph rather than by the flood hydrograph itself.

Then,

$$R \text{ (area under triangle)} = \frac{qT}{2} \times \frac{60 \times 60 \times 12}{5280 \times 5280} = 0.000775 qT$$

$$\text{or } q = \frac{R}{T \times 0.000775} = \frac{1290 R}{T} \quad (11)$$

From this formula, if T is assumed to be a constant, the peak flow will vary directly with the total run-off. Before considering whether this assumption that T is a constant is correct, this formula will be applied to certain flood hydrographs resulting from the flood of November, 1927. Figs. 13 and 14 show a number of such hydrographs together with the straight lines which balance up the actual hydrographs and determine the total flood period.

It will be noticed in the case of the Swift River at West Ware that the balancing lines include a horizontal line at the top, so that the simplified hydrograph is in the form of a trapezoid instead of a triangle. Hence, in this case the definition of "total flood period" must include the length of time in hours given by the horizontal line at the top as well as at the bottom of the simplified hydrograph.

This consideration of the flood hydrograph, including both the total volume of flood run-off and the "total flood period," should take into account only the flood run-off, excluding the base flow of the river which existed previous to the occurrence of the flood and continued independently of it. On the hydrographs horizontal dotted lines have been drawn to represent this base flow, and the "total flood period" has been determined by the intersection of the simplified hydrograph with this dotted line.

In Figure 14 the hydrographs are plotted according to gage heights rather than on a discharge basis. The use of gage heights for this purpose is approximate, as it assumes that the rating curves are straight lines. However, reference to Table 19 shows that these hydrographs, although plotted on a gage height basis, give a good approximation of the "total flood period." Due to the shape of the rating curves the "total flood periods" obtained from the gage height hydrographs are probably too long. Table 19 shows how closely the results obtained by using the formula $q = \frac{1290 R}{T}$ approach the actual results when the relationships between the peak flow, total run-off and "total flood period" are worked out graphically.

There are a great many more data available regarding the magnitude of storms, which determine the maximum possible run-off, than there

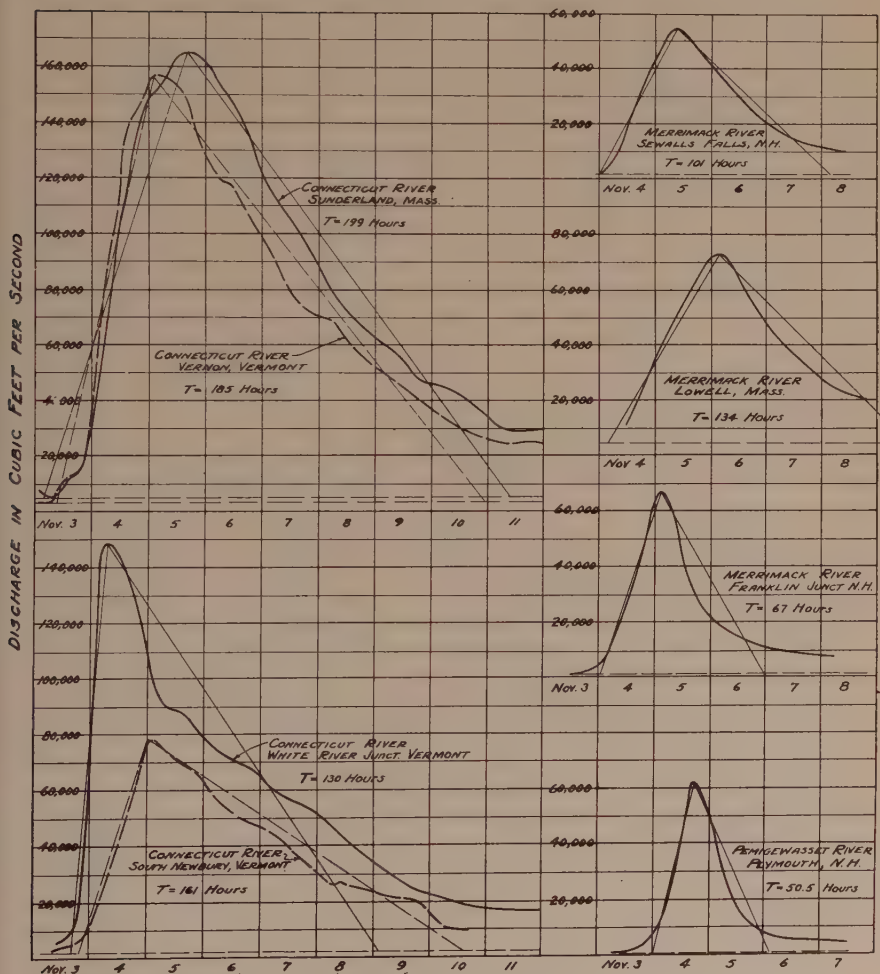


FIG. 13. — DISCHARGE HYDROGRAPHS OF VARIOUS STREAMS, NOVEMBER, 1927

are data regarding flood run-off. If the greatest flood on record at a given point was produced by a storm giving a total flood run-off of 4 inches from the entire drainage area, the flood which would be produced on that stream by a storm giving a total flood run-off of 6 or 8 inches, or

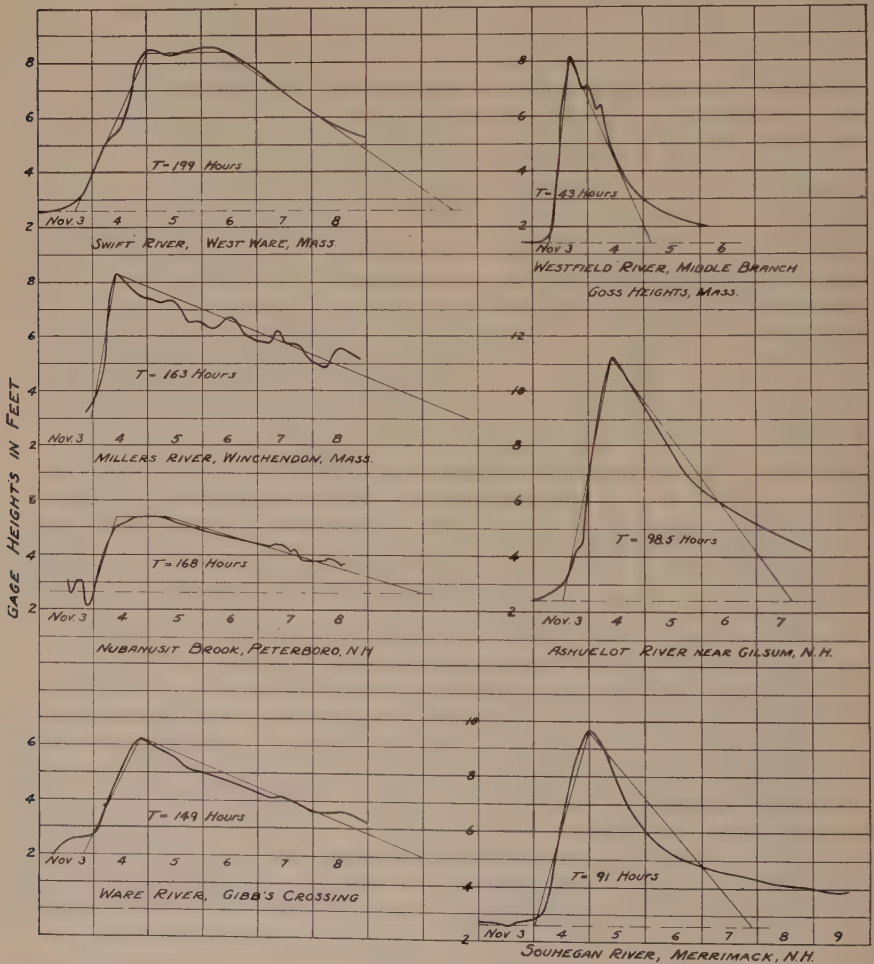


FIG. 14. — GAGE HEIGHT HYDROGRAPHS OF VARIOUS STREAMS, NOVEMBER, 1927

any other figure, could be estimated. It is believed that this method gives a more rational basis for determining how great a flood should be provided for than methods based on plotting drainage areas against run-off taken from data on streams of widely different drainage area characteristics or the use of flood frequency curves.

TABLE 19. — COMPARISON OF ACTUAL PEAK FLOW WITH PEAK FLOW DETERMINED GRAPHICALLY FROM "TOTAL FLOOD PERIOD" FOR 1927 FLOOD

RIVER AND STATION	Drainage Area in Square Miles	"Total Flood Period" obtained from Hydrograph in Hours	Flood Run-off from Drainage Area in Inches	PEAK FLOW* IN S. F./S. M.	
				Actual q	As obtained from "Total Flood Period"
		T	R	q	$1290 R$ $q = \frac{1290 R}{T}$
Connecticut River at Sunderland, Mass.	7,740†	199	3.26	21.3	21.1
Connecticut River at Vernon, Vt.	6,220†	185	3.45	24.9	24.1
Connecticut River at White River Junction, Vt.	4,040†	130	3.99	36.7	39.6
Connecticut River at South Newbury, Vt.	2,750†	161	3.34	28.4	26.8
Merrimack River at Sewalls Falls, N. H.	1,755†	101	2.56	31.4	32.8
Merrimack River at Lowell, Mass.	3,570†	134	2.06	20.4	19.8
Merrimack River at Franklin Junction, N. H.	935†	67	3.78	71.8	72.8
Pemigewasset River at Plymouth, N. H.	599†	50.5	3.96	104.0	101.0
Swift River at West Ware, Mass.	186	199‡	1.72	12.0	11.1
Nubanusit Brook at Peterboro, N. H.	54.3	168‡	1.70	18.6	13.1
Ware River at Gibb's Crossing, Mass.	201	149‡	1.88	14.1	16.2
Middle Branch of Westfield River, Goss Heights, Mass.	53	43‡	3.5§	111.0	105.0
Ashuelot River near Gilsum, N. H.	68.5	98.5‡	3.19	40.3	41.7
Souhegan River at Merrimack, N. H.	168	91‡	2.14	39.6	30.3

* The actual peak flows include the base flow; the computed results as obtained from the "Total Flood Period" do not.

† Reduced by amount of unproductive areas, or those having small regulated flow.

‡ "Total Flood Period" obtained from hydrograph of gage heights; due to shape of rating curve, these figures are probably too high.

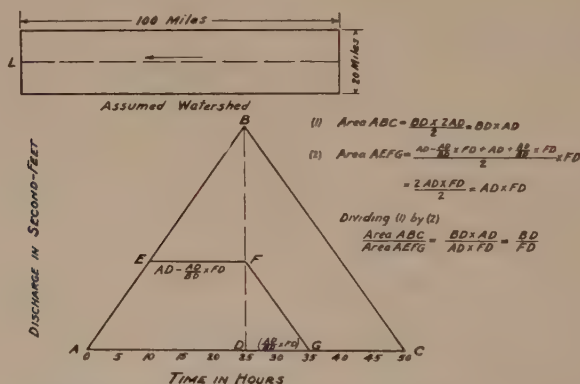
§ Estimated.

The correctness of the assumption that the peak flow varies directly with the total flood run-off, or that the "total flood period" is a constant for a given point on a stream, will now be considered.

Assume a rectangular drainage area such as the watershed outlined in Fig. 15, that the velocity is the same for all floods, and that the pondage is negligible. Assume it to be 20 miles wide and 100 miles long, and that the velocity of flood flow is 4 miles an hour. Under these conditions 25 hours will be required for precipitation which falls on the upper

end of the watershed to reach the lower end. This interval of time which it takes the water falling on the extreme part of the watershed to reach a given point under consideration is called the "concentration period" for the drainage area at that point.

If, now, precipitation is assumed to occur at a uniform rate during the concentration period of the watershed, that is to say, 25 hours in this case, the discharge at the lower end of the watershed will have the triangular shape shown by the lines ABC in Fig. 15. The area of the triangle ABC will represent the total flood run-off, and with 100 per cent run-off the peak flow BD will equal the rate of rainfall, since at the end of 25 hours the entire drainage area will be contributing its flow at the assumed uniform rate of rainfall.



Hydrograph of Flow at Point L for Streams of Uniform Intensity and Constant Velocity of Flow

FIG. 15. — DIAGRAM SHOWING EFFECT OF LENGTH OF STORMS UPON PEAK FLOWS

However, if the precipitation instead of continuing for 25 hours ceases at the end of 10 hours, the hydrograph at the lower end of the drainage area will assume the shape AEFG. It is apparent that area ABC is to area AEFG as BD is to FD, or that the total run-off in the two cases bear the same relation to each other as the corresponding peak flows. This relationship holds good for a storm of any duration up to the concentration period of the watershed. A storm which continues on beyond the concentration period for the watershed will not increase the peak flow any, for if the concentration period has been reached the unit run-off equals the rate of rainfall, and will continue the same until the storm ceases.

It is thus apparent, in considering the question of storms and floods, that the discussion should be limited to storms of less than the concentration period for the watershed in question. The longer the storms the lower the average rate of precipitation, so that the maximum flood-producing storm for any watershed is the maximum storm which can occur within the concentration period of the watershed. Furthermore, the "total flood period" for the assumed 25-hour flood is 50 hours, or double the concentration period of the watershed. In the case of the 10-hour storm, the trapezoid forming the hydrograph has a base of 35 hours and a top of 15 hours, or a total of 50 hours, the same as for the 25-hour storm. The same would hold good for a storm of any length short of the 25-hour concentration period.

Hence, for a *rectangular watershed* in which the velocity remains constant and the pondage is negligible, the "total flood period" remains constant and the peak flow will vary directly with the total run-off, provided the storm occurs within the concentration period of the drainage area.

In the case of a triangular-shaped drainage area, such as "C" in Fig. 6, it has been shown that an instantaneous storm would produce twice as large a peak as would a storm of the same total run-off, with the precipitation falling over a time equal to the concentration period of the drainage area. From this it follows that the "total flood period" in a given stream, in so far as possible, should be based upon a storm approaching its concentration period. A storm of shorter duration will give too small a "total flood period" on a triangular-shaped drainage area. If the shape is so irregular that the flood is produced by a very heavy storm of short duration, then the "total flood period" should, if possible, be determined from such a flood; but ordinarily the maximum flood to be expected will come from the maximum rainfall anticipated during the concentration period of the stream.

The effect of changes in velocity and channel pondage in a given stream for floods of varying magnitude must also be considered. The natural tendency is for both the velocity and pondage to increase as the size of the flood increases. If these two factors increase in the proper ratio, the "total flood period" will remain unchanged and the flood peak will vary directly with the total volume of the flood. Although definite data are meager, they indicate that in many cases the mean velocity tends to approach a constant, as is shown in Fig. 9 of the Connecticut River at Sunderland, or in some cases actually decreases. Fig. 9 is a fair representation of what occurs in relatively flat stretches of rivers; with rising stages the stream stretches out over wider and wider

areas. On such streams the hydraulic radii do not necessarily continue to increase, while the friction due to trees and obstructions does. On the other hand, the velocities undoubtedly do increase with rising stages

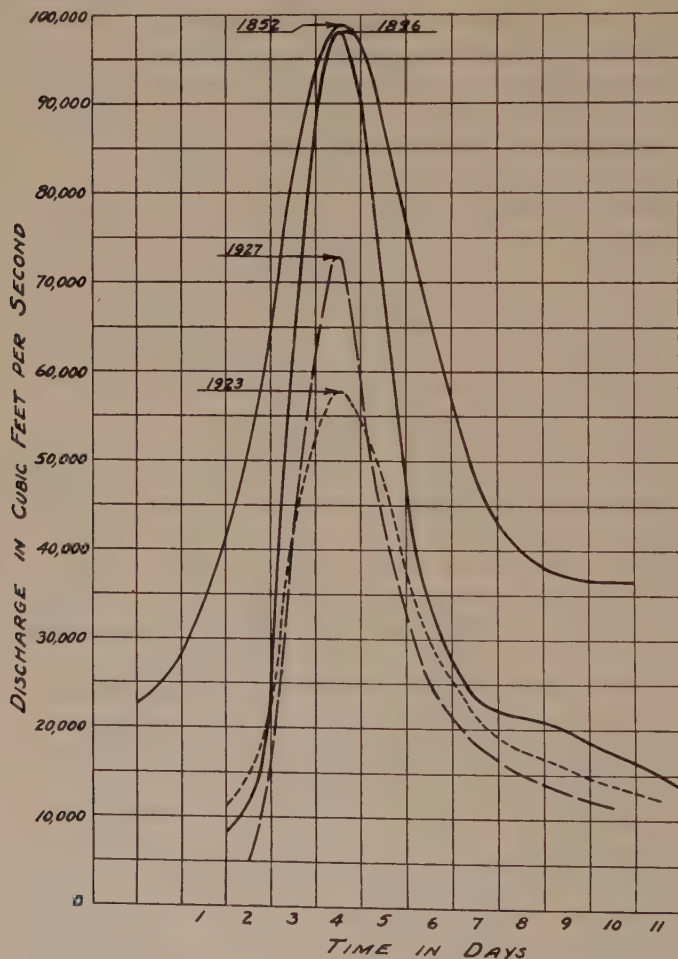


FIG. 16.—FLOOD HYDROGRAPHS, MERRIMACK RIVER, AT LOWELL, MASS.

on steep streams confined to narrow valleys. In such cases a larger storm means a shorter "flood period."

Hence, for flat streams with large channel pondage, the "total flood period," and consequently the peak run-off, increases less rapidly

than the total flood discharge, while on steep streams with little channel pondage "the total flood periods" decrease, and consequently the flood peaks increase more rapidly than the total run-off.

Fig. 16 shows the hydrographs of several floods on the Merrimack River at Lowell. These floods show variations in the "total flood period," as follows:

TABLE 20.—VARIATIONS IN FLOOD PERIOD—MERRIMACK RIVER AT LOWELL

DATE OF FLOOD	Peak Flow in Second Feet	"TOTAL FLOOD PERIOD"	
		Days	Hours <i>T</i>
1852	99,000	10.4	249
1896	98,500	7.1	170
1923	57,900	6.6	159
1927	73,000	6.4	153

This table brings out the effect of different kinds of storms. The 1852 flood on the Merrimack was caused by rain over a period of about 10 days, combined with melting snow. The precipitation and the melting snow were not confined to the concentration period of the watershed. Hence, the resulting hydrograph gives a "total flood period" that is too long. Melting snow adds a complication to most spring floods that makes the determination of the "total flood period" from spring floods less accurate than in the case of most fall freshets. On the other hand, the 1927 flood was caused by a short, severe storm which in most parts of the Merrimack watershed lasted but 4 to 8 hours, and thus would tend to shorten the "total flood period" as compared with a storm of equal total run-off extending over the concentration period. This table indicates that the "total flood period" of the Merrimack River above Lowell is about 170 hours. It also illustrates the fact that judgment and general knowledge of the storms producing floods must be applied in determining the "total flood period."

From the available data it is believed that the variations in "total flood periods" are not more than 50 per cent either way. It is believed that a general consideration of the characteristics of a stream and its behavior during previous floods of varying magnitude should permit the estimate of a reasonable "total flood period" for greater floods than those on record. If a 3-inch flood run-off on a flat stream with large channel pondage gave a "total flood period" of 6 days and a peak unit flow

of 27 second feet per square mile, then a 6-inch flood run-off on the same stream would doubtless give a "total flood period" longer than 6 days and a peak flow of less than 54 second feet per square mile. A steep stream with little channel pondage giving a "total flood period" of 2 days and a peak flow of 80 second feet per square mile for a total flood run-off of 3 inches would give a "total flood period" of less than 2 days and a peak in excess of 160 second feet per square mile for a total flood run-off of 6 inches. In all cases the period of the storm must be less than the concentration period of the watershed, or else an allowance must be made for the precipitation and run-off which takes place beyond such limits.

The concentration period for any point as used in storm sewer design is defined as the time required for the water to flow from the most remote part of the watershed to it. The preceding discussion of drainage area characteristics has followed this definition. The existence of pondage and storage, however, will tend to retard the time of the peak, and makes it more difficult to determine the concentration period. Practically, the concentration period for any point can probably best be taken as the length of time from the beginning of a short, severe storm to the arrival of the peak flow at the given point. Data covering the exact period of rainfall are not available for many storms previous to that of November, 1927, but the time of the beginning of the storm may be taken as the time when the first increase in flow at the uppermost tributary in the drainage area occurs. Hence the concentration period may be taken as the time between the beginning of the increase in flow at the uppermost tributary and the time of the peak flow at the point in question.

With a uniform rainfall distribution, the longer the storm the longer the time required for the stage to reach its peak at a given point. The concentration period is desired in order to ascertain the length of storms producing serious floods, but a precise determination is not required. For example, on a stream where from, say, 30 to 60 or 70 hours or more are required before the peak is reached, if the concentration period is determined for a short, severe storm of, say, 2 to 4 hours' duration, an allowance for a longer storm and for the effect of pondage should probably be made by adding between 6 and 24 hours.

No two storms are alike. A 4-inch rainfall may be distributed over a period of time from 2 to 48 or even 84 hours. In this discussion, it has been assumed that storm rainfall occurred at a uniform rate, which, of course, is never true. The hydrographs resulting at a given point on a stream from storms of different distribution will vary in shape. How-

ever, there appears to be a marked resemblance in the shape of the hydrographs for all storms that are limited in duration to the concentration period. Storms extending beyond the concentration period will entirely alter the shape of the flood hydrograph. The prolonged effect resulting from melting snow frequently introduces this complication, and many large floods in New England, particularly on the larger rivers, have occurred in which the storms causing them, combined with the effect of the melting snow, have been longer than the concentration period of the drainage area.

Determination of Flood Run-off

Following a rainstorm, a certain amount of the water runs off over the surface of the ground, and within a period of a few days flows down the river system and into the ocean, part is held back in lakes and ponds, another and very important part is absorbed by the soil, while a small amount is evaporated. Part of the water absorbed by the soil appears in the streams and rivers in the form of ground water. It is not possible to draw any sharp dividing line between the time when the surface run-off ceases and the ground water flow from a given storm begins. Doubtless at the lower end of the watershed the ground is contributing a certain amount at the time of the arrival of the peak of a flood, so that the flood peaks themselves contain ground water flow which may continue for weeks and even months after the flood.

However, it is the amount of run-off during the flood period rather than the total eventual yield of a storm that is important in considering flood problems. Some arbitrary point must be chosen as the end of the flood in arriving at a figure for the flood run-off. After an examination of the flood hydrographs of numerous New England floods, it was decided to take as the period of flood run-off the time between the beginning of the rise and the time at which the flow falls to within 2 second feet per square mile of the initial flow. This ordinarily gives a run-off nearly equal to the total flood run-off, but in extreme cases, where the peak flow is not more than a few second feet per square mile, it is necessary to include a longer period. It was found in the development of "flood characteristic curves" (see page 392) that taking a period in hours equal to four times the square root of the drainage area in miles would usually cover the period of flood run-off excepting for sluggish streams, where a longer period was required. Both of these criteria have been used in the computation of the depth of flood run-off in this report. The total flow during the period of flood run-off is thus made up of two parts: first, the base flow which existed previous to the occurrence of the storm,

and second, the flood run-off or the additional flow which may be considered as superimposed upon the base flow.

In the case of floods which occur following a long dry period, the base flow may be neglected without materially affecting the results. However, in the case of spring floods in which the flood run-off resulting from the storm is superimposed upon a large base flow, usually caused by the melting of snow, the base flow assumes considerable importance, and should be deducted. It has been previously shown that the peak flows tend to vary directly with the total run-off, neglecting the base flow. Actually, the peak flow will be greater than that caused by the flood run-off by an amount equal to the base flow. Hence, the flood run-off corresponding to the peak flow will lie between the total flood run-off, including the base flow, and the actual flood run-off. This flood run-off corresponding to the actual peak which includes the base flow will be called the "equivalent concentrated flood run-off." This has been determined on the basis that the "equivalent concentrated flood run-off" bears the same relation to the flood run-off (base flow eliminated) as the actual peak flow bears to the flood peak above the base flow.

A GENERAL FORMULA FOR NEW ENGLAND

From the previous discussion of drainage area characteristics it is evident that the flood period T will vary directly with the length of the stream, other conditions being the same. For similarly shaped drainage areas the "flood period" tends to vary directly with the square root of the drainage area.

The size of the drainage area has been used so extensively in flood formulæ that its square root has been taken rather than the length of the stream. Table 21 gives the total flood period and the drainage area for several New England streams. The last column gives the unit flood period in hours for each mile represented by the square root of the drainage area.

TABLE 21. — GIVING RELATION OF FLOOD PERIOD TO DRAINAGE AREA

RIVER AND STATION	TOTAL FLOOD PERIOD T		Gross Drainage Area A (Square Miles)	$\sqrt{A}$ (Miles)	$C_f = \frac{\text{Col. (3)}}{\text{Col. (5)}}$ (Hours per Mile)
	Days	Hours			
(1)	(2)	(3)	(4)	(5)	(6)
1. Merrimack River at Lowell .	7.1	170	4,097	64.	2.66
2. Connecticut River at Sunderland	8.9	214	8,000	88.7	2.41
3. Penobscot River at West Enfield	8.2	195	4,720*	68.7	2.84
4. Kennebec River at Waterville .	3.2	78	3,030*	55.0	1.42
5. Androscoggin River at Rumford Falls	4.1	98	995*	31.5	3.11
6. Pemigewasset River at Plymouth	2.1	51	615	24.8	2.06
7. Westfield River at Westfield .	1.3	32	496	22.3	1.44
8. Nashua River at Nashua .	6.5	157	539	23.2	6.77
9. Souhegan River at Merrimac ..	2.9	69	168	13.0	5.30
10. Murchison Farm, Tennessee .	.026	.62	.175	.42	1.48

* Uncontrolled drainage area.

It will be observed from this table that C_f , the unit flood period in hours per mile, for these streams varies from 1.42 for the Kennebec River at Waterville up to 6.8 for the Nashua River at Nashua. If all streams were hydraulically similar the unit flood period in hours per mile would be constant, but with their different characteristics it varies materially. This variation is a direct measure of the flood producing characteristics of a stream.

Substituting $T = C_f \times \sqrt{A}$ in the fundamental formula

$$q = \frac{1290 R}{T} \quad (12)$$

it becomes $q = \frac{1290 R}{C_f \sqrt{A}} \quad (13)$

This can be simplified by substituting a new coefficient, C_F , in place of $\frac{1290}{C_f}$. This coefficient C_F will be called the flood characteristic of a stream. The formula then becomes —

for the unit peak flow, $q = \frac{C_F R}{\sqrt{A}}$ second feet per square mile (14)

for the total peak flow, $Q = C_F \sqrt{A} R$ second feet (15)

In this formula the coefficient C_F is a measure of the flood producing characteristics of a drainage area, A is the size of the drainage area in square miles, and R is the flood run-off in inches depth on the drainage area. These three factors take care of the drainage area characteristics of a watershed, its area, and the magnitude of the storm, respectively. From equations (12) and (14) it is evident that $\frac{C_F}{\sqrt{A}} = \frac{1290}{T}$, or that $C_F = \frac{1290}{T} \sqrt{A}$.

Table 22 gives the values of C_F for a number of New England streams having a wide range of drainage area characteristics, for the Miami River, and for a small brook on Murchison's Farm in Tennessee having a drainage area of 112 acres, or .175 square mile. These values of C_F vary from a maximum of about 1,000 for the flashy streams down to 169 for a sluggish stream having a large volume of channel storage, such as the Nashua River above Nashua, and even down to a value as low as 83 for the Swift River and 68 for the Concord River at Lowell. For New England streams it appears that 1,000 may be taken as the maximum value of C_F with values in the mountainous regions of between 500 and 1,000. For average conditions outside the mountainous regions it will vary from 100 to 500, while for extremely flat streams, where the volume of pondage is relatively large, the value of C_F may be less than 100.

COMPARISON WITH OTHER FORMULÆ

This formula $Q = C_F \sqrt{A} R$ is the same as the Meyer formula, $Q = C \sqrt{A}$, with the exception that the run-off factor R has been added to designate the magnitude of the flood under consideration. Jarvis* modified the Meyer formula by choosing a coefficient of $C = 10,000$ as a maximum, and then referred particular floods to this maximum on a percentage basis.

Major Pettis,† in his treatise, "A New Theory of River Flood Flow," developed the formula $Q = 328 P W^{5/4}$ in which W is the average width of the drainage area in miles and P is a rainfall factor. He limits the use of this formula to drainage areas of between 1,000 and 10,000 square miles, and calls attention to special exceptions to it. He shows how his formula is an extension of the Meyer formula, with the exception that he provides for different rainfalls, and substitutes the width for the

* See "Flood Flow Characteristics," C. S. Jarvis, Transactions, American Society of Civil Engineers, Vol. 89, p. 985.

† Maj. C. R. Pettis, "A New Theory of River Flood Flow," 1927.

TABLE 22. — VALUES OF C_F IN $Q = C_F \sqrt{A} R$

RIVER AND LOCATION	DRAINAGE AREA IN SQUARE MILES		FLOOD USED AS BASIS FOR DETERMINING C_f				Q in Second Feet	C_f IN $Q = C_f \sqrt{A} R$	
	Total	Uncon- trolled	Date	CONCENTRATED STORM RUN-OFF R IN INCHES		Based on Total Drainage Area		Based on Total Drainage Area	Based on Un- controlled Drainage Area
				Based on Total Drainage Area	Based on Un- controlled Drainage Area				
Merrimack River at Lowell	4,097	3,570	1896	3.17	3.62	98,500	486	454	
Connecticut River at Sunderland	8,000	7,694	1927	3.47	3.60	165,000	536	530	
Penobscot River at West Enfield	6,600	4,720	1909	2.16	3.02	94,420	538	455	
Kennebec River at Waterville	4,270	3,030	1901	2.20	3.12	155,600	1,080	907	
Androscoggin River at Rumford Falls	2,090	995	1927	1.70	3.42	44,700	601	415	
Pemigewasset River at Plymouth	615	—	1927	4.00	—	62,000	626	—	
White River at West Hartford	695	—	1927	6.00*	—	140,000	885	—	
Westfield River at Westfield	496	—	1927	3.00†	—	42,500	635	—	
Nashua River at Nashua	539	421	1927	1.64*	2.1*	7,275	191	169	
Concord River at Lowell	374	284	1927	1.60	2.1	2,400	78	68	
Swift River at West Ware	186	—	1927	1.93	—	2,180	83	—	
Souhegan River at Merrimac	168	—	1927	2.1	—	6,650	244	—	
Miami River near mouth	3,937	—	1913	8.2	—	384,000	748	—	
Miami River at Dayton	2,525	—	1913	7.0‡	—	250,000	712	—	
Murchison Farm, Tennessee	.175	—	1918§	.92	—	333	866	—	

* Estimated.

† Estimated equivalent flood run-off. Total run-off, 4.37 inches, but storm was longer than concentration period.

‡ Estimated equivalent flood run-off. Total run-off, 8.76 inches, but storm was longer than concentration period.

§ April 19.

square root of the drainage area which provides for variations in the distribution of drainage area. He considered the following three formulæ:

$$Q = 813 PW$$

$$Q = 328 PW^{5/4}$$

$$Q = 132 PW^{3/2}$$

He chose the second, $Q = 328 PW^{5/4}$ as conforming most closely to the plotted points on his diagram. It will be observed that the first of these, $Q = 813 PW$, is the same in form as $Q = C_F \sqrt{A} R$, with the exception that the latter substitutes the flood run-off in inches in place of the rainfall, and uses the square root of the drainage area in place of the width.

The proposed formula is also similar to the "Rational Method" used in the design of sewers, $Q = C i A$, where i is the intensity of rainfall in inches per hour, C a coefficient representing the per cent of run-off, and A the area of the drainage area in acres. The intensity is taken as the maximum average rate to be expected during a period equal to the concentration period of the drainage area. If this concentration period in hours is represented by T_c , then it is evident that the run-off in inches $R = C i T_c$, or $C i = \frac{R}{T_c}$. Substituting $\frac{R}{T_c}$ for its equivalent $C i$ in the rational formula, the formula becomes $Q = \frac{A R}{T_c}$. Furthermore, for similar drainage areas the concentration period T_c will vary directly with the linear dimensions, or directly with the square root of the drainage area. Hence, by introducing a different coefficient and changing A from the area in acres to the area in square miles the formula becomes $Q = \frac{C_F A R}{\sqrt{A}} = C_F \sqrt{A} R$, which is the formula that the committee has proposed.

FLOOD CHARACTERISTIC CURVES

If both the time and quantity of flow in a flood hydrograph are divided by the square root of the drainage area, a new hydrograph will be obtained for a similar flood on a similar drainage area of 1 square mile. If the quantities of flow in the new hydrograph for 1 square mile are again divided by the number of inches of flood run-off resulting from the flood, a hydrograph for a 1-inch flood on 1 square mile will be obtained. This will be called a "Characteristic Flood Curve." By the use of this method the characteristic flood curves of different streams can be plotted together and the differences in the drainage area character-

istics shown graphically. Furthermore, the hydrographs of various floods at a given point on the same river can be similarly treated to indicate graphically the behavior of the stream under different types of storms and varying sizes of floods. It is also evident that the peak point of the flood characteristic curve gives the coefficient C_F in the formulæ (14) and (15).

If parts of a drainage area are completely controlled by storage so that they contribute little or nothing to the flood run-off, then these

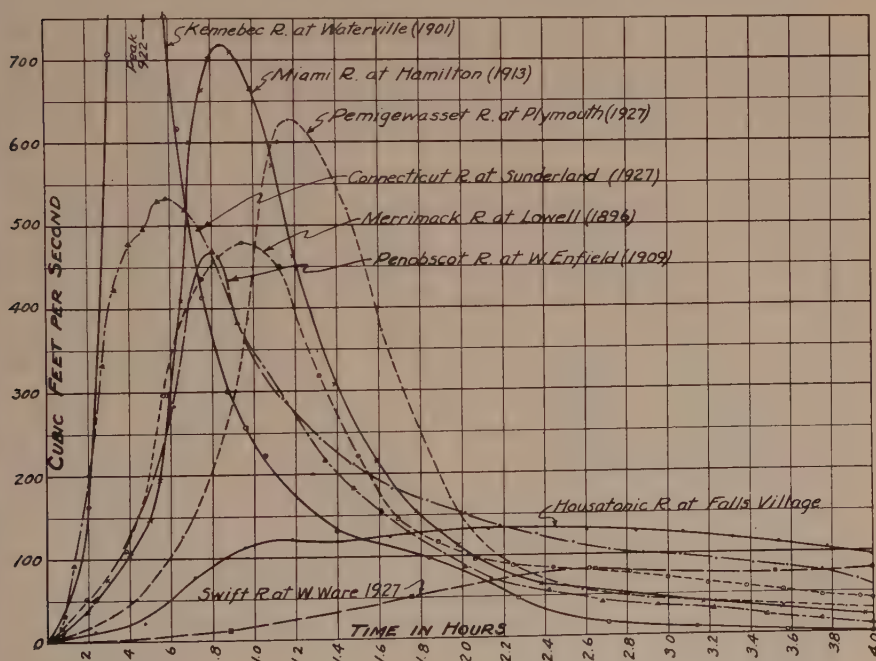


FIG. 17. — CHARACTERISTIC FLOOD CURVES FOR VARIOUS STREAMS

controlled areas should be deducted from the gross drainage area and the net drainage area contributing to the flood used. The gross drainage area will give a higher flood characteristic than that obtained with the net drainage area. For various percentages of controlled area, the differences will be as shown in Table 23. In view of all the uncertainties connected with flood problems, it is not believed that it is worth while to distinguish between the gross drainage area and the net drainage area, excepting in such cases as the controlled area amounts to more than 25 per cent of the total drainage area.

TABLE 23. — EFFECT OF USING GROSS DRAINAGE AREA INSTEAD OF NET DRAINAGE AREA UPON VALUE OF C_F

PER CENT OF AREA CONTROLLED										Effect on Value of C_F (Per Cent)
12.5	.	.	.	.	.	.	.	.	.	+ 7
25	.	.	.	.	.	.	.	.	.	+16
50	.	.	.	.	.	.	.	.	.	+40

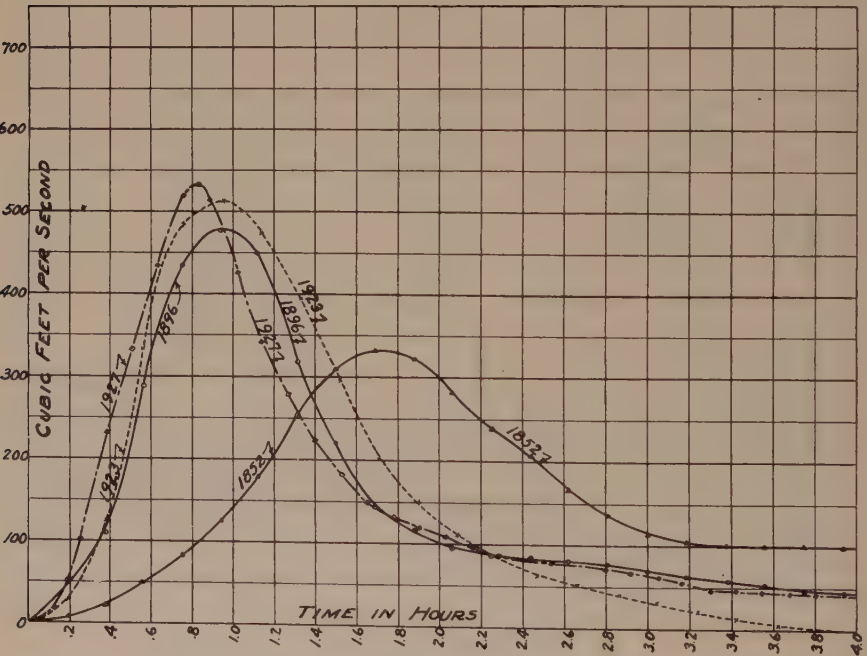


FIG. 18. — CHARACTERISTIC FLOOD CURVES FOR DIFFERENT FLOODS ON MERRIMACK RIVER AT LOWELL,^a MASS.

Fig. 17 shows the characteristic flood curves for a number of New England streams, and also for the Miami River at Hamilton for the 1913 flood. Fig. 18 shows the characteristic flood curves of several floods on the Merrimack River at Lowell. The flood characteristic curve for the 1852 flood at Lowell shows the effect of a storm considerably longer than the concentration period.

The flood characteristic curves in Fig. 17 bring out the wide differences that exist in drainage area characteristics. The Swift River at West Ware, with but 186 square miles of drainage area, has a flood characteristic, C_F , of 83, which corresponds to a "total flood period," T , of 210 hours on this size of drainage area. If the topography of this watershed was hydraulically similar to that of the Kennebec River, with its rapid rate of run-off giving a value of C_F approaching 1,000, then the "total flood period" would be 17.6 hours instead of 210 hours.

In making up these curves it was found that a period corresponding to 4 hours on the characteristic curve would include substantially all of the flood run-off, excepting on sluggish streams where a longer period was required. This 4-hour period on the characteristic curve means a period of four times the square root of the drainage area for the stream itself. In a few cases it was found that additional precipitation occurred before this period was ended. In such cases an estimate has been made of what the flow would have been if the subsequent rains had not occurred. The possible errors involved in such estimates are insignificant compared to the shape of the flood characteristic curve as a whole.

A certain amount of judgment must be used in choosing the point of beginning of the flood curve. If the precipitation during the storm fell at a uniform rate this difficulty would not arise, but if the main storm is preceded by a lesser one, then the point of beginning must be chosen with particular reference to the flood hydrograph as a whole. On that account, the length of time from the beginning of the flood to the peak on the flood characteristic curves may be made to vary by possibly one-quarter of an hour, depending upon the point selected as the beginning of the flood. Differences in selecting the point of beginning of the flood have but little effect upon the peak obtained, except as it alters the value assumed for the base flow. In general, the selection of a base flow that is too high will raise the peak of the flood characteristic curve and increase the value of C_F , while the assumption of too low a figure for the base flow will reduce the peak and the value of C_F .

THE RUN-OFF, R

The run-off is equal to the precipitation plus melted snow and less amounts absorbed and evaporated. Great floods are due to great storms caused by an unusual combination of circumstances. They are the results of unusually heavy precipitation occurring at such a time that the watersheds will yield a high run-off. A few degrees' difference in temperature may determine whether a storm will result in a flood or a

blizzard. The condition of the ground previous to a storm will determine the amount of the precipitation that is absorbed by the soil. A 5-inch or 6-inch storm, which under some conditions might run off completely and produce a great flood, may under other conditions be largely absorbed by the soil. We have no knowledge as to when these unusual combinations of conditions will occur.

On the other hand, the experience of eastern United States during the last one or two centuries gives a basis for estimating the maximum conditions to be expected. The heaviest storms on record combined with the worst conditions of snow, temperature and ground water should afford the basis for estimating what might be called the maximum possible flood. Between the floods which are more or less certain to recur at various intervals and the maximum flood is a wide belt of uncertainty.

The volume of flood run-off in inches on the drainage area, R , to be expected in New England can be estimated either on the basis of rainfall or stream flow records.

Determination of "R" from Rainfall Records

In considering what values of R are to be anticipated in New England, it is important to bear in mind that the storm must be limited to the concentration period of the drainage area in order for the previous analysis to apply. Thus, the smaller the drainage area the shorter the storms to be studied, and in determining R from rainfall records it is first necessary to ascertain the length of storms to be considered. Column 6 of Table 21 gives the relation between the total flood period, T , in hours, and the square root of the drainage area, which for various New England streams varies from 1.42 to 6.80, with an average value of about 2.25. The ratio between the concentration period and the total flood period on a number of streams shows that this ordinarily varies between .3 and .45, with a value of about .4 for average conditions. Using a unit flood period of 2.25 hours and a concentration period of about .4 of the total flood period, the concentration period for average conditions is about .9 of an hour for each mile represented by the square root of the drainage area. Using this average figure, the concentration period for various sizes of drainage areas is given in Table 24. For flashy streams the concentration period will be shortened by 25 to 50 per cent, while for sluggish streams it may become 50 to 100 per cent longer, and even more in extreme cases where storage occurs.

TABLE 24. — DRAINAGE AREA v. CONCENTRATION PERIOD

DRAINAGE AREA (SQUARE MILES)	Average Concentration Period (Hours)
10	2.84
100	9.00
500	20.2
1,000	28.4
2,000	40.2
4,000	56.9
8,000	79.8
10,000	90.0

In Table 25 the concentration period has been taken from Table 24, the rainfall to be expected with a 50-year frequency within the concentration period has been taken from Table 9, and this rainfall has been combined with an assumed per cent of run-off to give an estimate of the run-off in inches.

TABLE 25. — ESTIMATED 50-YEAR RUN-OFF BASED ON CHESTNUT HILL RAINFALL RECORDS

[Average Conditions]

DRAINAGE AREA (SQUARE MILES)	Concentration Period (Hours)	50-Year Rainfall in Concentration Period (Inches)	Assumed Percentage of Run-off (Per Cent)	Run-off (Inches)
10	2.84	3.48	85	2.96
100	9.00	5.00	80	4.00
500	20.2	6.40	70	4.48
1,000	28.4	7.09	65	4.61
2,000	40.2	7.91	60	4.75
4,000	56.9	8.83	55	4.85
8,000	79.8	9.75	50	4.88

This table is based on broad assumptions, and no allowance has been made for a decrease in precipitation as the area increases. However, it is believed that it is reasonable to conclude from it that a storm confined to the concentration period producing a flood run-off of between $2\frac{1}{2}$ and $3\frac{1}{2}$ inches is likely to occur with a frequency of about once in 50 years.

Table 26 is made up from Table 24 combined with the rainfall to be expected in 100 years, given in Table 12 (page 349).

TABLE 26. — RAINFALL *v.* AREA RELATIONS DURING CONCENTRATION PERIOD

DRAINAGE AREA (SQUARE MILES)	Concentration Period (Average Conditions) (Hours)	100-Year Rainfall (Interpolated from Table 12) (Inches)
500	20.2	8.0
1,000	28.4	8.2
2,000	40.2	7.7
4,000	56.9	7.9
8,000	80.5	6.8
10,000	90.0	6.9

Thus it will be seen that an estimated rainfall of from 6.8 to 8.2 inches in depth may be expected within the concentration period of watersheds from 500 to 10,000 square miles with a 100-year frequency. This rainfall would ordinarily mean a run-off of from 4 to 6 inches.

Under the discussion of Rainfall-Area Relations and Frequency an estimate was given for the rainfall-area relations likely in any 20-year period, and also of the maximum rainfall-area relations. These are given in Table 27, with the addition of the flood run-off in inches assuming a 66.7 per cent of run-off. These estimated relations were as follows:

TABLE 27. — AREA-RUN-OFF RELATIONS

Rainfall in inches	5	6	7	8	9	10	11	12
Flood Run-off 66.7 per cent factor .	3.3	4	4.7	5.3	6	6.7	7.3	8
Maximum area covered by rainfall (square miles) .	25,000	15,000	8,000	2,500	1,000	500	250	50
Area likely to be covered in any 20-year period (square miles)	15,000	8,000	4,000	2,000	600	50	—	—

This table would indicate a somewhat greater figure for floods than the other tables. However, it does not take into consideration the length of the storms, and, as previously shown, a given amount of run-off resulting from a storm longer than the concentration period will not produce as high peak flows as the same amount of run-off resulting from a storm that is of shorter duration than the concentration period. Consequently, some reduction should be made from the above table, particularly for the smaller drainage areas, to allow for this element of time.

From these various tables the following general conclusions are drawn:

1. An occasional flood run-off, R , of 3 inches within the concentration period is to be expected in New England for drainage areas from 10 square miles up to 10,000 square miles. Information is too limited to definitely state the frequency; but in general it appears that such a flood should be looked for with a frequency of between 25 and 75 years, or, say, once in 50 years.

2. A rare flood run-off, R , of 6 inches within the concentration period is to be expected for all drainage areas of from 10 square miles to 10,000 square miles in New England. The frequency of such floods is probably somewhere between 50 and 200 years.

3. The maximum flood run-off, R , is probably not over 8 inches.

4. From the flood history of New England it appears that storms producing heavy run-off are likely to occur with greater frequency in those regions having a high annual rainfall than in those territories with a low annual rainfall. However, the information available is not sufficient to determine whether there is any direct relation which can be formulated.

Determination of "R" from Run-off Records

The known flood history of most of New England extends over a period approaching 150 years. Data available previous to 1900 are rather meager, but the early records of rainfall and river heights are probably sufficient to indicate fairly closely the relative magnitude of the earlier floods as compared with those of the last half century. Table 28 has been made up to show the magnitude of the two largest floods since 1896 on the five main river systems draining New England. It appears that seven storms have produced the two largest floods on each of these five rivers which have occurred since 1896. Furthermore, the largest flood on each of these rivers since 1896 is substantially equal to any other flood within the known history of these streams.

The flood of November, 1927, produced the largest flood on record on the Connecticut River, and on many of the upper tributaries of the Merrimack and Androscoggin Rivers. It also produced substantial but not excessive floods on the lower reaches of both the Merrimack and Androscoggin Rivers. The flood of February 29 to March 3, 1896, caused by rain and melting snow, produced the largest floods in this period on the lower Merrimack River and the lower Androscoggin River. In December, 1901, a combination of melting snow and rain produced the greatest flood within the history of the Kennebec River.

TABLE 28. — FLOOD STUDIES ON

	MERRIMACK RIVER AT LOWELL, MASS.		CONNECTICUT RIVER AT SUNDERLAND, MASS.	
Total drainage area (square miles)	4,097	3,979	8,000	
Controlled drainage area (square miles)	527	527	306	
Uncontrolled drainage area (square miles)	3,570	3,452	7,694	
Flood of	Feb. 29- Mar. 10, 1896	Nov. 4- Nov. 14, 1927	Mar. 26- Apr. 3, 1913	Nov. 3- Nov. 18, 1927
Days of flood run-off	11	11	9	15
Base flow	5,000	3,700	45,000	8,500
Flow from controlled drainage area	475	540	—	100*
On total drainage area:				
Total run-off (inches)	3.51	2.43	3.49	3.86
Flood run-off (inches)	3.01	2.06	1.60	3.26
Base run-off (inches)	.50	.37	1.89	.60
Equivalent concentrated storm run-off in inches	3.17	2.17	2.40	3.44
On uncontrolled drainage area:				
Total run-off (inches)	4.03	2.67	—	4.01
Flood run-off (inches)	3.45	2.30	—	3.39
Base run-off (inches)	.58	.37	—	.62
Equivalent concentrated storm run-off in inches	3.62	2.30	—	3.57
Peak flow:				
From total drainage area	98,500	73,000	135,000	165,000
From total drainage area S. F./S. M.	24.0	18.4	16.9	20.6
From uncontrolled drainage area (with flow from controlled drainage area deducted)	98,025	72,460	—	164,900
From uncontrolled drainage area (with flow from controlled drainage area deducted) S. F./S. M.	26.4	21.0	—	21.4
Peak flood flow (with base flow deducted)	93,500	69,300	90,000	156,500
Computed total flood period in hours	170†	153†	184†	216†
Value of flood characteristic C_F	486†	533†	628†	536†

* Estimated.

† Based on total drainage area.

‡ Based on uncontrolled drainage area.

VARIOUS NEW ENGLAND RIVERS

CONNECTICUT RIVER AT WHITE RIVER JUNCTION, VT.	CONNECTICUT RIVER AT --		PENOBSCOT RIVER AT WEST ENFIELD, ME.		KENNEBEC RIVER AT WATER- VILLE, ME.		ANDROSCOGGIN RIVER AT RUMFORD FALLS, ME.	
	ORFORD, N. H.	SOUTH NEWBURY, VT.						
4,120 80 4,040	3,300 80 3,220	2,830 80 2,750	6,600 1,880 4,720		4,270 1,240 3,030		2,090 1,095 995	
Nov. 3- Nov. 11, 1927	Mar. 25- Mar. 31, 1913	Nov. 4- Nov. 10, 1927	Sept. 28- Oct. 10, 1909	Apr. 27- May 9, 1923	Dec. 15- Dec. 25, 1901	Apr. 29- May 7, 1923	Mar. 1- Mar. 6, 1896	Nov. 3- Nov. 9, 1927
9 6,000 25*	7 25,400 25*	7 5,000 25*	12 4,200 2,280	13 40,000 3,000	11 3,500 1,200	9 20,000 250	6 1,800 1,400	7 2,580 2,000*
4.23 3.74 .49	3.12 1.11 2.01	3.46 3.01 .45	2.36 2.07 .29	5.92 2.98 2.94	2.47 2.15 .32	5.19 3.62 1.57	1.45 1.26 .19	1.92 1.60 .32
3.90	1.99	3.21	2.16	4.04	2.20	—	1.32	1.70
4.32 3.82 .50	3.19 1.13 2.06	3.57 3.09 .48	3.29 2.89 .40	8.28 4.17 4.11	3.48 3.03 .45	7.32 5.10 2.22	2.73 2.64 .09	3.52 3.36 .16
4.03	2.02	3.26	3.02	5.64	3.12	2.75*	2.67	3.42
148,000 35.9	57,200 17.4	78,000 27.6	96,700 14.7	153,000 23.2	156,800 36.7	136,000 31.8	39,000 18.7	46,700 22.4
147,975	57,175	77,975	94,420	150,000	155,600	135,750	37,600	44,700
36.7	17.8	28.3	20.0	31.8	51.4	44.9	37.8	44.9
142,000	31,800	73,000	92,500	113,000	153,300	116,000	37,200	44,120
140† 593†	149† 498†	150† 456†	190† 466†	224† 455†	77† 907†	171§ 414§	89† 458†	98† 415†

§ Based on uncontrolled drainage area. The storm was of much longer duration than the concentration period. Hence values of T and C_F are much distorted.

The greatest flood on the Penobscot River was caused by rain and melting snow in April and May, 1923. This same storm caused the second largest flood on the Kennebec. The second largest flood on the Penobscot at West Enfield was produced by heavy rains in September, 1909, while the second largest flood on the Connecticut River, within the period 1896 to date, occurred in March, 1913.

An examination of this table shows that within this period each of these rivers has had an equivalent concentrated flood run-off of from 3 to 4 inches, based on the uncontrolled drainage area, with a flood exceeding 5 inches on the Penobscot in 1923. The 1896 flood on the Merrimack River at Lowell, with a flood run-off of 3.62 inches, was probably equaled or slightly exceeded in 1852 and 1785. The flood of November, 1927, on the Connecticut, at Sunderland, of 3.60 inches was substantially equaled in the lower part of the river in 1854 and 1801. The flood of November, 1927, on the Androscoggin at Rumford Falls, with an equivalent concentrated flood run-off of 3.42 inches, was probably not reached by the flood of 1896 nor that of 1805. The 1901 flood on the Kennebec River at Waterville of 3.12 inches is the greatest within a period of 50 years, while the record on the Penobscot is relatively short.

These five rivers represent an average flood history of at least 100 years each, making a total of over 500 years. During this period each river system has had floods of from 3 to 4 inches, with an average frequency of about 50 years. The greatest flood was that of 1923, on the Penobscot, of 5.64 inches. Hence, this table tends to confirm the conclusions drawn from the rainfall records, — that a flood of about 3 inches is to be expected about once in 50 years, and that a flood of 5 to 6 inches may occur. The great storm of 1913 produced a flood run-off of over 8 inches on the Miami River, with a drainage area of 4,000 square miles. It is fair to assume that a similar storm and flood run-off are possible in New England, and it is largely on this basis that 8 inches has been taken as the maximum to be expected in New England.

The flood history of the smaller streams is not as complete as on some of the larger rivers, yet sufficient information is available to indicate the general trend. The 1927 flood on the Westfield River produced a run-off of 4.37 inches at the United States Geological Survey gaging station below Westfield. The storm causing it lasted somewhat longer than the concentration period, and the peak stage corresponded to a concentrated storm run-off of from 3 to $3\frac{1}{2}$ inches. This flood was exceeded by those of December, 1878, and October, 1869. In 1878 the peak flow at West Springfield was estimated at 46,000 cubic feet per second; in 1927 the flow was estimated at 38,000 cubic feet per second.

This would indicate a flood run-off of 4 inches or more in 1878. The 1869 flood fell between the other two. Hence, the Westfield River has experienced three floods of over 3 inches within the last 60 years.

The Deerfield River at Charlemont had a total flood run-off of about 6 inches on the drainage area below Davis Bridge in 1927, estimated as an equivalent concentrated flood run-off of between 4 and 5 inches. The peak flow was 36,000 or 200 second feet per square mile. In July, 1915, before the construction of the Davis Bridge Reservoir, there was a storm producing a flood run-off of 2.61 inches and a peak flow of 45,000 cubic feet per second, or 135 second feet per square mile from the 332 square miles of drainage area below the Somerset Reservoir. The 1869 flood on the Deerfield was probably greater than this, though definite information is not available.

Most of the streams in western Connecticut, western Massachusetts and Vermont had a flood run-off of from 4 to 7 inches during the 1927 flood. In many cases the storm was longer than the concentration periods, and the peaks were lower than would have been the case had the amount of run-off occurred within a shorter period. Attention has been called to several small tributaries of some of the Vermont streams which have experienced higher floods in other years due to a more intense rainfall of lesser total depth. In the 1927 storm the greater part of the rainfall in Vermont occurred during a period of 18 hours, which is about the concentration period of many of the main streams entering Lake Champlain on the west and the Connecticut on the east. This resulted in some remarkably high floods on these streams, the most striking of which was the 140,000 second feet on the White River at West Hartford, a unit flow of 200 second feet per square mile from the 695 square miles. On many of these streams the flood of October, 1869, was the highest previously known.

The Chicopee River presents a somewhat different situation. As a result of the 1927 storm a flood run-off of 1.93 and 1.99 inches, respectively, occurred on the Ware and Swift Rivers, which are two of its main tributaries. The rainfall over the Chicopee drainage area in November, 1927, was less than on the western side of the Connecticut River. The greatest flood on record on the Chicopee was that of October, 1869, regarding which no definite data are available. Since the average rainfall and run-off on the Chicopee watershed are less than that in the Berkshire and Green Mountain region, it is probable that floods of 3 or 4 inches run-off are of less frequent occurrence on the Chicopee than on the streams which have been discussed. It also appears from Table 14 that the absorptive capacity of the soil on the Chicopee drain-

age area, particularly in the case of the Swift River, is greater than for streams like the Westfield River, so that in the long run a storm of the same size will produce a smaller volume of flood run-off on the Chicopee.

The Pemigewasset River at Plymouth, N. H., with a gage height of 27.5 feet, had an equivalent concentrated flood run-off of 4.31 inches in November, 1927. This was the greatest flood on record at this point, although possibly not much higher than the flood of October, 1869. The November, 1927, storm was of but few hours' duration above Plymouth, and was entirely within the concentration period. In April, 1895, the river reached a stage of 24 feet, while in March, 1896, 20 feet was the high point. According to general reports the 1869 flood was higher than that of 1895 and 1896, but not as high as that of 1927. All four floods presumably had a flood run-off of between 3 and $4\frac{1}{2}$ inches.

These data, though approximate, tend to confirm the previous conclusions of 3 inches for the occasional flood, and 6 inches for the rare flood. Probably 8 inches may be assumed for the maximum flood on the basis of the 1913 flood on the Miami River.

Fig. 19 shows the curves obtained from the formula proposed by the Committee for certain values of C_F and R plotted with several other well-known flood formulæ; also points for the 1927 flood are shown. Most of the empirical formulæ are apparently based on a flood run-off of some 5 to 6 inches on an average type of stream, or 3 to 4 inches on a flashy stream. The maximum curve shown by Jarvis for his modified Meyer formula ($Q=10,000\sqrt{A}$) is above the highest of the proposed curves. In comparing floods, he referred them to this maximum curve on a percentage basis. His coefficient of 10,000 is equivalent to a C_F of 1,000 and a flood run-off of 10 inches in the committee's formula, while the maximum suggested by the Committee is given by a coefficient, C_F , of 1,000 with a run-off, R , of 8 inches, corresponding to a coefficient of 8,000 for the Meyer-Jarvis formula.

Spillway Capacity for Storage Reservoirs

This method of considering flood flows on the basis of the total volume of flood run-off affords an approach to the problem of the necessary spillway capacity for a storage reservoir. A storage reservoir may take care of floods in different ways. Sufficient storage may be provided which will always be available to absorb the flood, or sufficient discharge capacity provided to discharge the flood waters as rapidly as they flow into the reservoir. Usually a combination of both methods is employed. If a storage reservoir has a given discharge capacity, then the

important factor is the total run-off for such a period as the inflow exceeds the discharge capacity. During such a period the volume of water discharged, plus the increase in storage, will equal the total run-off. If for an assumed set of conditions the water would rise above the highest safe water level, then either the discharge capacity must be increased or more storage provided by raising the dam. This effect of the storage lengthens the concentration period so that it is likely to be several days or several weeks. In such cases it may be necessary to provide for a flood run-off greater than the 8 inches which has been designated as a maximum.

SUMMARY

For flood flows for New England, the Committee recommends the following formulæ:

$$q = \frac{C_F R}{\sqrt{A}} \text{ for unit run-off in second feet per square mile}$$

$$Q = C_F \sqrt{A} R \text{ for peak flows in second feet}$$

Where R is the flood run-off in inches on the drainage area, A , the size of the drainage area in square miles, and C_F a flood coefficient depending upon the flood characteristics of the stream.

The flood coefficient, C_F , must be determined from previous flood hydrographs with allowance for any changes resulting from the contemplated flood. The characteristics of the drainage area, and its behavior during previous floods of varying magnitude as reflected by flood hydrographs, afford the best basis for such an allowance. The values of the coefficient, C_F , vary between 1,000 as a maximum, and less than 100 as a minimum. For most streams in the mountainous regions it is between 500 and 1,000; for average types of streams outside the mountainous regions the value varies between 100 and 600; for flat streams with relatively large channel pondage, between 60 and 100. In most cases the hydrograph resulting from a single short storm of moderate intensity should be sufficient to permit a reasonable determination of the value of C_F . Where no flood hydrographs are available, the value of C_F must be estimated from the general conditions on the drainage area compared with other drainage areas where the values of C_F can be determined from flood hydrographs.

The recommended values for the run-off in inches, R , are as follows:

For occasional floods, 3 inches (25 to 75 year frequency).

For rare floods, 6 inches (say 50 to 200 year frequency).

For maximum flood, 8 inches (200 years and upwards).

An 8-inch flood would probably mean doubling the largest flow on record on the main river systems of New England except the Penobscot.

In concluding this discussion on flood formulæ for New England, the Committee would emphasize the fact that the flood situation at any point on any stream presents a problem of its own. No general formula can be of universal application. It is only by special study of all the data, and the conditions for the point under consideration, and comparison with floods on similar streams that the best results can be obtained. The flood hydrographs of previous floods, when studied in connection with the nature and type of the storms producing these floods, are believed to afford the best information for solving the problem.

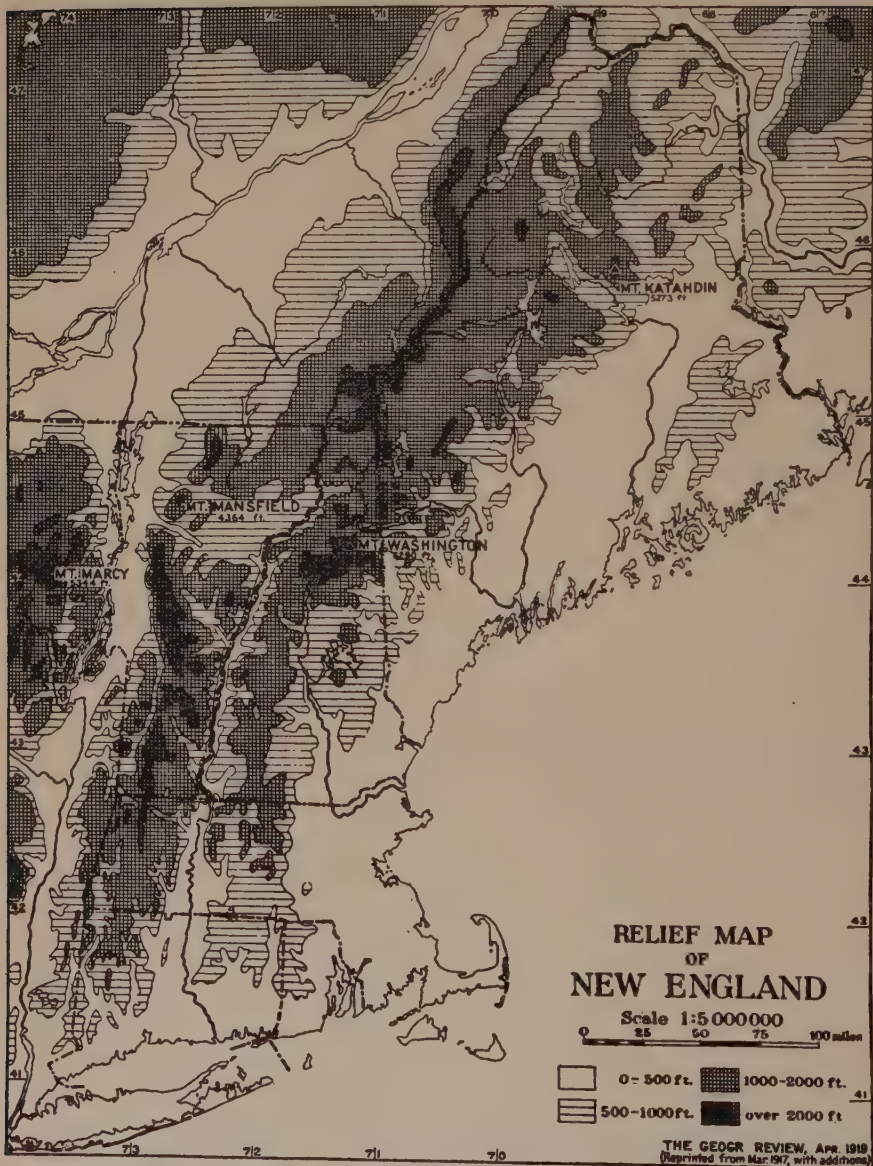
The two important fundamental factors are, first, the total volume of flood run-off, expressed by R , and second, the shape of the flood hydrograph, which determines the relation between the total volume of flood run-off and the peak flows. It is hoped that the methods developed here constitute a forward step, and it is believed that they will be of help in the solution of flood problems.

Chapter VI. — Flood Characteristics of New England Streams

GEOLOGICAL FEATURES

New England, more than any other region of its size in the eastern states, has drainage basins of extraordinary variety. Floods gather and discharge in a manner that seems almost haphazard. Nature operates with flood equations composed of so many variable factors that one can only imperfectly analyze them on the basis of observed data.

This irregular concentration and delivery of floods in New England may be traced to an oddly irregular terrain — the result of an unusual combination of geologic structures and circumstances. Even the patchy rainfall of New England is due in part to irregularities of surface relief, for measurements of precipitation at mountain stations in New Hampshire since 1927 have confirmed the earlier evidence that in rugged sections, like the White Mountains, lateral deflection of air currents and local updrafts occasioned by topographic barriers result in much diversity of rainfall. The uneven snow-cover on these hills and mountains, subject to spring thaws, and the irregular distribution of forest and open country are two more variables in the problem. So at the very start the run-off is likely to become concentrated independently at a large number of points.



Courtesy of the Geographical Review published by the American Geographical Society of New York.

PLATE VI. — RELIEF MAP OF NEW ENGLAND

But in particular, the idiosyncracies of drainage in New England may be traced to geologic structure and glacial history. Here a small-scale patchwork of miscellaneous rocks, only locally simple in structure, wrinkled and crushed and shot through with masses of granite, was worn deeply by long exposure to weather and stream erosion during long ages before the glacial period; its soils were removed by the glacier, and in place of them was spread a new mantle of drift, so uneven and so ill fitted to the hills and valleys that it scarcely conceals the roughened rocky surface. The river systems were badly disorganized. As the ice sheet withdrew into Canada, lakes occupied many of the valleys, and the sea overspread the coast of Maine and New Hampshire, and invaded the Champlain valley — for the region then stood lower. Soon, a widespread upwarping of the ice-freed area brought New England to its present position, slanting more steeply seaward than before; the sea retired from the Champlain valley and the Maine coast; the lakes drained out of the Merrimack and Connecticut valleys and their tributaries, and rivers commenced cutting down their channels through drift-filled valleys along somewhat new lines. This task has brought them into contact with hidden ledges at hundreds of places on their courses, where they are caught, as on thresholds, while swinging widely on softer ground upstream.

In the following paragraphs the dominant characteristics of geologic structure and glaciated surface will first be briefly described; then their bearing on flood discharge will be indicated by describing certain typical examples of drainage basins in New England, where these conditioning factors appear in different combinations.

Granites and other massive rocks of igneous origin occupy large areas in New England. (See Fig. 19a.) Because they have no bedding planes or other particular lines of structural weakness, they tend to create broad, shapeless topographic forms without definite linear expression. Mountains lie in irregular groups instead of ranges, and though their peaks may be sharp (as is the case, for example, with Mt. Cho-corua) they usually are rounded. Drainage patterns in such areas are diverse and difficult to classify. A much larger part of New England is occupied by metamorphic rocks, like slates, schists and quartzites, which differ considerably in resistance to weathering, and so produce relief features that have a strong grain. Drainage lines show some adjustment to the weaker belts of rock, and ridges to the stronger ones, though severe glaciation has somewhat obscured the detail. So we find hills of the "Berkshire type" trending north and south or northeast-southwest, parallel to rock structure, from one end of New England to the other.

They show best in the interior where the upland stands highest and streams have cut deepest. The N.N.E.-S.S.W. trend of the Connecticut valley between Vermont and New Hampshire is one of the chief cases of adjustment.

Large areas in New Hampshire and Maine, and smaller ones in southern New England, are occupied by crystalline schists and gneisses

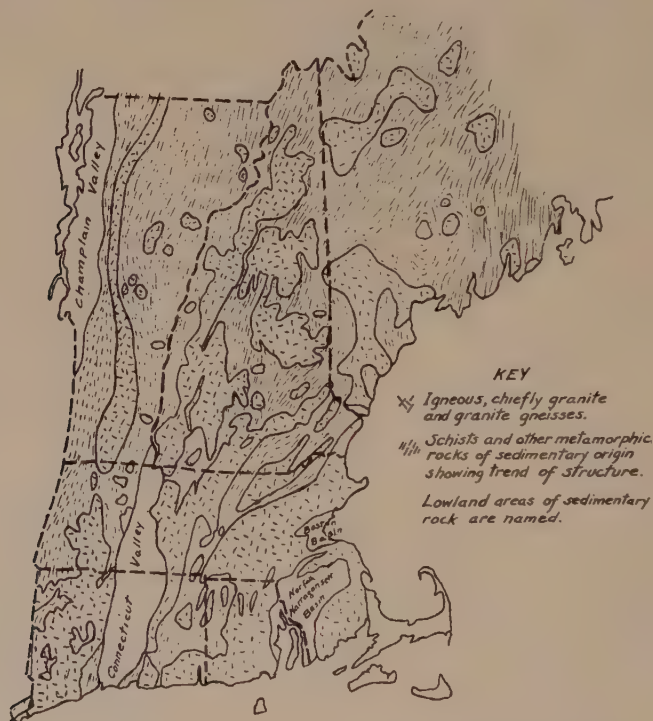


FIG. 19A. — BED ROCK GEOLOGY OF NEW ENGLAND
(GENERALIZED)

that are almost as strong as granites, and so nearly uniform that in these areas the grain of the topography is obscure, and details of slope depend primarily on glacial scour. In Vermont, on the other hand, tightly compressed, wrinkled strata, punctured in only a few places by granites, extend in straight belts of nearly uniform width for long distances; and since they differ markedly in strength they form parallel ranges of mountains and foothills. Thus the hard gneissic core of the Green Mountains, stoutly buttressed by inclined beds of quartzite on

both flanks, nearly bisects the State. Rivers like the Missisquoi and Winooski, while not heading on the mountain axis but crossing it, display structural control in the rectangular pattern of their main branches. Draining through the mountain range by deep gaps, they descend westward to the low limestone valley of Lake Champlain, on slopes made doubly steep by late-glacial emergence from the Champlain Sea. East of the Green Mountains an enormous area of lime-bearing slates and schists in Orange and Windsor Counties, draining into the Connecticut, forms hills and ridges that have symmetrical round profiles and deep loamy soils, in marked contrast to the diversified rocky ridges, hills and hollows of the New Hampshire area.

In Maine, belts of slates and schists are much wider, and run diagonally across the state, northeast-southwest. Their topographic effects are less conspicuous in the interior than on the ragged coast, where a rising sea has half drowned the ridges and valleys. Massachusetts is crossed by continuations of these squeezed and folded strata that reach from northern New England right down to Long Island Sound. The proportion of granite is even greater in Massachusetts and Connecticut than in New Hampshire. Over all New England the ancient folds of these Appalachian Mountains have been planed down to a low rolling upland. The hilltops of today seem to register the reduction of the whole surface in Mesozoic time to a featureless plain, which has since been uplifted and carved by streams in the variety of ways described.

Lowlands of considerable size have been eaten out by rain and rivers in areas where the rock structure is weak. The great Champlain valley marks the surface of a soft limestone, less affected by mountain folding and more soluble than the rock structures that underlie the Green Mountains. The Connecticut valley lowland (in western Massachusetts and Connecticut) has developed on an extensive area of soft red shales and sandstones. Tilted layers of hard trap rock protruding between the weaker strata have been sharpened into narrow mountain ridges. Nearer the coast, small areas of slate make the less conspicuous Boston and Narragansett basins. In central New Hampshire, Lake Winnepesaukee lies in the bottom of a broad lowland where granite of an exceptionally weak granulated type has succumbed to weathering and glacial abrasion, while the harder schists and igneous structures of surrounding ranges stood fast. Another lowland of this sort lies in the Penobscot basin in Maine.

The movement of the continental ice sheet from Canada across New England was roughly seaward, — southeastward in Maine, New Hampshire and eastern Massachusetts, and southward elsewhere. Soils

that must have accumulated thickly before the Ice age seem to have been wholly scoured off and the rocky surface deeply worn. In some places ledges were scrubbed smooth; in others, roughened by the extraction of blocks and boulders. The ice did not cut out new hills and valleys, but it strongly modified them, leaving them more irregular than before. In the lowland of Lake Winnepesaukee, where the sheet was very thick and the rock was weak, granite hills were scoured heavily into arched elliptical forms that closely resemble the drift hills or drumlins of Boston; and long irregular hollows were cut in the soft granite floor of the lowland, which are now conspicuous parallel arms of the lake. Over New England generally, however, the influence of glaciation is perhaps less evident than that of rock structure. Even the moderate irregularity of the rock surface thus produced has had a marked effect on flood discharge.

The unevenness of surface was increased greatly by the mantle of drift left by the ice sheet. Stony upland soil — the ground moraine of the glacier — was smeared all over the hills and valleys in patches so variable in thickness and so irregular in outline that progress of run-off toward drainage lines is quite erratic. Lakes, ponds, bogs, meadows and well-drained slopes lie spread out in confusion where the surface in preglacial time probably had orderly valley systems. As the ice melted away, also, gravels, sand and silt were washed from it into the valleys, accumulating there in a great variety of forms, — as eskers, kames, kame terraces, deltas, and in deep-water lake beds of silt as much as one or two hundred feet thick. So while the valleys were aggraded with relatively porous material, capable of absorbing much water, the hillsides were mantled with boulder clay that sheds water easily.

While the ice border was receding into Canada, New England was upwarped or tilted seaward at a rate of several feet to the mile. There seems to have been several upward movements or broad bulgings of the earth's crust, repeated at long intervals. The effects on drainage were pronounced. Long, deep lakes in the Merrimack and Connecticut valleys were drained. Rivers proceeded to cut down their channels rapidly through the soft valley deposits, stopping in this process only where they encountered bedrock in their channel floors or banks. In this way New England rivers, caught on ledges at hundreds of points, came to have an abundance of water power. Small streams furnished the numerous mill sites in colonial days, and larger rivers the greater sites for manufacturing centers and hydroelectric power of more modern times. Upstream from each rock threshold rivers swing from side to side in meanders, trimming back the soft terrace bluffs until they meet

protecting ledges, and creating those wide flood plains known to Indians as the upper and lower "Coos" on the Connecticut, and to early settlers as "intervalles." These wide flood plains afford enormous channel storage at time of flood, when water may overflow them as much as 5 or 10 feet, and spread sidewise 1 or 2 miles, while the sand and gravel structure soaks up a little of the flood water. Each great meadow ends down valley more or less abruptly in a narrow pass or bottleneck, at the ledge or dam that constitutes the threshold. When flooded, the river consists of a series of basins of different areas, depths and shapes, which fill and discharge as water passes in and out of them down steeper segments. These variable characteristics alone on a river like the Merrimack would make flood flow complex; but the actual condition is still more extreme, for instead of a single thread of drainage we have many threads joining from opposite sides, and all of them possess the bottleneck and basin character. The influence that these features have on the movement of floods down valley is profound.

An equally important effect of glaciation is the diversity of patterns of drainage. This can be seen best on a map that shows only the bare outlines of the rivers and their branches, unconcealed by contour lines, lettering or other details. All the larger rivers, such as the Merrimack, Saco and Androscoggin, are oddly deformed, unsymmetrical aggregations of stream lines full of kinks, crooks, undersized and oversized branches, that dodge in and out among the hills as aimlessly as an old New England country road or cowpath.

So, through a long and varied series of geological events, that starts with ancient mountain folding and erosion and ends with continental glaciation, upwarping and rejuvenation, it has come about that the river systems of New England display that extreme variety of tributary patterns, that odd assortment of stony and sandy soils, those interrupted gradients, and that abundance, yet ill-arranged grouping, of ponds and other natural storage areas that make floods gather and run off with great disorder.

DRAINAGE AREA DESCRIPTIONS

In order to show the application of the principles outlined in Chapters IV and V, and to indicate the flood characteristics of some of the larger New England river systems most subject to flood damage, the following brief description of certain drainage areas is given. These discussions include a general description of the topography, a brief summary of characteristic flood behavior, including the time element, and, where possible, estimates of the value of the flood coefficient C_F in the

formula $Q = C_F \sqrt{A} R$. These approximate estimates have been given as two limiting figures between which the coefficient may vary somewhat.



FIG. 20. — LAKE CHAMPLAIN DRAINAGE AREA

These descriptions are of a general nature and undoubtedly do not cover all conditions and types of floods. A complete analysis of the flood characteristics of a stream would require numerous flood hydrographs on both the main stream and its more important tributaries for different

types and sizes of floods. Complete information of this sort is not available on any of the river systems. An attempt has been made to point out the salient features from the available data.

LAKE CHAMPLAIN DRAINAGE BASIN

The Lake Champlain drainage basin in Vermont (see Fig. 20) includes the Missisquoi, Lamoille and Winooski Rivers and Otter Creek. These four streams have a total drainage area of about 3,600 square miles, or nearly 40 per cent of the area of the state. They drain the western slopes of the Green Mountains with numerous mountain peaks between 3,000 and 4,500 feet elevation.

Winooski River (1,076 square miles) has a length of about 90 miles, with steep slopes of some 8 to 12 feet per mile. About one-half of the 400 feet of fall below Montpelier is developed at six dams with short mill ponds and steep slopes between the successive developments. The river is bordered by a range of mountains between it and the Connecticut drainage basin, and lower down the river itself passes through gaps in two high mountain ranges. The drainage basin is fan-shaped with the peak flows on the main tributaries synchronizing to a considerable degree. The river was characterized by the great rise in the water level during the 1927 flood, thus creating considerable channel pondage. The depth on the dams of the Green Mountains Power Corporation, near Winooski Gorge, 27 feet, Essex Junction, 19.3 feet, Bolton Falls about 17 feet, and in the stretches of river above these dams, varied between 25 and 57 feet. Flood coefficient at Essex Junction (1,034 square miles), 550-600; at Montpelier (420 square miles), 500-600.

Lamoille River (710 square miles) is also about 90 miles long and has an average slope of about 14 feet per mile, although in the 20 miles above Fairfax Falls (529 square miles) the fall is not over 15 feet. The drainage area is not as compact as the Winooski. The rise in water level during the 1927 flood varied between 15 and 20 feet along the main river, except near the town of Johnson, where a rise of 38 feet was reached. Flood coefficient, 500-600.

Missisquoi River (885 square miles), about 85 miles long, also resembles the Winooski and the Lamoille, the slopes ranging from 8 to 12 feet per mile, excepting in the 12-mile stretch of the river in Canada, which is quite flat. The drainage area is more compact than the Lamoille, but somewhat less so than the Winooski. The headwater rose to about 15 feet on the dams during the 1927 flood, with the tailwater rising between 20 and 38 feet. Flood coefficient, 500-600.

Otter Creek (925 square miles), with its long and narrow drainage area, runs from south to north, with the southern part of the Green Mountains forming the easterly side of its watershed. The slopes of the main river are much flatter than for the rivers described above, and below Rutland the fall is largely concentrated between long flat stretches, affording sluggish flood characteristics. Large areas of marshes and flats provide much greater natural storage than is to be found on either of the other three rivers draining into Lake Champlain. During the 1927 flood, the main river rose between 7 and 15 feet, with the exception of just above and below Proctor, where the rise was between 17 and 24 feet. The tributaries are small, but have a quick run-off. Flood coefficient of main river below Rutland, 100-200; on quick run-off tributaries, 300-500.

Flood Control Requirements

Some 3 to 6 billion cubic feet of flood storage would largely control flood flows on each of these river systems. Of these four rivers, flood conditions on the Winooski are the most severe, and 4 to 6 inch floods will inevitably produce great damage.

CONNECTICUT RIVER

The Connecticut River (11,345 square miles), with its long and narrow drainage area, extends from the Canadian line to Long Island Sound, a distance of nearly 400 miles (see Fig. 21). It drains the west side of the White Mountains (Mt. Washington, El. 6,290) and the eastern slopes of the Green Mountains in Vermont, and the Berkshires in Massachusetts.

Third Connecticut Lake is at approximately elevation 2,200. More than half of the fall occurs in the upper 35 miles above West Stewartstown. In the next 60 miles between there and Gilman, the fall of 200 feet is mostly concentrated in the vicinity of Lyman Falls, with long, wide stretches of intervals above and below. An additional 390 feet of fall occurs in the Fifteen Mile Falls section between Gilman and McIndoe's Falls, leaving the river about 420 feet above sea level at McIndoe's Falls, a distance of 272 miles from the Sound. The greater part of this fall is concentrated at seven dams with long mill ponds. Under 1927 flood conditions the water rose between 10 and 15 feet on the dams, and between 25 and 35 feet at the upper ends of the mill ponds, creating a large volume of channel pondage, particularly above the dams at Bellows Falls and Holyoke. (See Table 18.)

The flood characteristics of the Connecticut River are exceedingly diverse.* The drainage area above Gilman (1,540 square miles) is very sluggish on account of the wide intervals and flat slopes between there and West Stewartstown. In 1927 the peak at the Waterford Bridge, a few miles below Gilman, occurred 16 hours after the peak had passed Sunderland, Mass.

Between McIndoe's Falls and White River Junction the peak for concentrated storms is determined primarily by the flow of the Passumpsic and the Lower Ammonoosuc Rivers, both steep streams draining mountainous regions with rapid run-off characteristics. In 1927 the peak from this section of the river preceded that from up river by about 40 hours, and was itself about 30 hours behind the peak at White River Junction, caused by the White and Ottauquechee Rivers.

The river from White River Junction down can be treated as a unit. For concentrated storms the highest water occurs at White River Junction as a result of the rapid run-off from the White and Ottauquechee Rivers. This peak travels down the river at a rate of between 2 and 4 miles per hour, arriving at Bellows Falls, Holyoke and Hartford about 8 hours, 34 hours and 65 hours, respectively, after leaving White River Junction. The tributaries on the west side, including the White, Ottauquechee, West, Deerfield, Westfield and Farmington Rivers, are all streams of rapid run-off and high flood characteristics. Tributaries from the east, including the Mascoma, Sugar, Ashuelot, Millers and Chicopee Rivers, all have sluggish flood characteristics, due to long stretches of flat slopes, considerable storage, and much channel pondage. The peaks of the quick run-off tributaries below the Ottauquechee on the west all precede the main river peaks by 15 to 65 hours, while the peaks of the sluggish tributaries on the east side are all well spread out, and either synchronize with or come later than the main river peaks.

The length of the storms has an important bearing on the behavior of the Connecticut River floods. For storms of less than 30 to 40 hours' duration the main river peaks at White River Junction are determined by the flood run-off from the White and Ottauquechee Rivers, and pass down the river without much change in the volume of peak flow. As a result of the large volume of channel pondage the flood hydrographs become flatter going down river, with the front part of the flood wave contributed by the early discharge of the quick run-off tributaries from the west, and with the back part of the wave supplied by the belated

* See J. W. Goldthwaite, "The Gathering of Floods in the Connecticut River System," *The Geographical Review*, July, 1928, pp. 428-445.

flow from the main river above White River Junction, and by the sluggish tributaries from the east. With longer storms, the differences in the time of flood peaks are reduced. With a 4-day storm the flow from the upper watershed will reach White River Junction while the White River is still contributing its full flow. With 5 or 6 day storms it is probable that nearly all parts of the river system would be at peak flow at substantially the same time. Flood coefficients of Connecticut River at Gilman, 250; at South Newbury, 450; at White River Junction, 600; at Sunderland, 550.

Connecticut River Tributaries

Passumpsic River (484 square miles). — Basin broadly fan-shaped with steep slopes averaging 16 feet per mile in 35 miles, and branches that synchronize well. Fall concentrated in part at numerous dams. The flood coefficient at St. Johnsbury, 500–600.

Lower Ammonoosuc River (395 square miles). — Steep slopes, flashy tributaries fed from White Mountains, drainage area long, narrow and crooked. Flood coefficient, 450–600.

White River (710 square miles). — Steep slopes averaging 14 feet per mile for 50 miles, with very little concentrated fall and channel pondage. Drainage area square, with nearly simultaneous concentration of four parallel main branches. Flood coefficient at West Hartford (695 square miles), 800–900.

Mascoma River (206 square miles). — Diversified drainage area with several lakes and reservoirs that regulate flow, particularly the largest, Mascoma Lake, midway down the system. Fall below Mascoma Lake concentrated between flat slopes. Flood coefficient, 100–150.

Ottawaquechee River (215 square miles). — Very steep slopes with small part of fall concentrated at dams with small mill ponds. Drainage area long and narrow, flashy tributaries. Flood coefficient at Dewey's Mills (208 square miles), 400–500.

Sugar River (266 square miles). — Considerable storage at extreme headwater in Lake Sunapee (drainage area, 60 square miles). Total fall great, most of it is concentrated, with flat slopes in between. Flood coefficient, 100–200.

West River (404 square miles). — Drainage area long and narrow with steep slopes and little pondage. Flood coefficient, 400–600.

Ashuelot River (440 square miles). — Large area, branches fail to synchronize. Numerous lakes and ponds. Flat slopes between concentrated falls. Flood coefficient at Hinsdale, 100–150.

Millers River (394 square miles). — Porous soil. Fall concentrated

between long stretches of river, with flat slopes and much channel pondage. Flood coefficient at Erving (372 square miles), 150-200.

Deerfield River (665 square miles). — The Davis Bridge and Somerset Reservoirs fully control 184 square miles. Steep, rocky slopes. Flood coefficient at Charlemont, 500-600, based on uncontrolled drainage area.

Chicopee River (721 square miles). — Characterized by concentrated falls and flat slopes on both tributaries and main river. Flood coefficient, 70-100.

Westfield River (515 square miles). — Drainage area fan-shaped. Steep slopes. Some channel pondage in lower reaches. Flood coefficient at gaging station below Westfield, 600-700.

Farmington River (about 600 square miles). — Upper parts of drainage area steep with rapid run-off. Much of fall in State of Connecticut concentrated at numerous dams with long mill ponds. River long and drainage area rather narrow. Flood coefficient at Rainbow (581 square miles), 200-300.

Flood Control Requirements

It is evident that storage on the White River is the key to the control of flood flows on the Connecticut River below White River Junction. From 4 to 6 billion cubic feet of capacity available for absorbing flood run-off located on the White River would substantially lower the peaks on the main river for concentrated storms. Flood storage on the other rapid run-off tributaries would help, particularly by leaving the channel pondage on the main river available for reducing the peak flows.

MERRIMACK RIVER

The Merrimack River (5,015 square miles) is formed by the junction of the Pemigewasset (1,013 square miles) and the Winnepesaukee (480 square miles) at Franklin, N. H. (see Fig. 22). The Pemigewasset, properly the main stem of the system for one-third of its length, although not called by the name Merrimack, has its sources in the southern part of the White Mountains, with the Franconia Range (El. 5,259) located between its east and west branches. The Pemigewasset has a total drop of about 1,500 feet in the 38 miles between its source and Plymouth, and an additional 200 feet in the next 30 miles, between Plymouth and Franklin, a large part of this 200 feet being concentrated at Bristol and at Franklin.

The flood waves normally start with the Pemigewasset River, and travel at the rate of between 2 and 5 miles per hour. The concentration period at Plymouth is between 15 and 20 hours. The peak at Franklin Junction comes about 10 hours after the peak of the Pemigewasset at Plymouth; at Garvins Falls, 22 to 24 hours after the Plymouth peak; and at Lowell, between 30 and 40 hours after the Plymouth peak. For concentrated storms the peak at Franklin Junction is materially reduced between there and Garvins Falls by the large volume of channel storage. (See Table 18.)

The principal tributaries below Franklin Junction have sluggish characteristics and moderate peak flows. The peak on the Souhegan precedes the main river peaks by about 12 hours, while peaks on the Nashua and Suncook Rivers occur at approximately the same time as highwater on the Merrimack. The Contoocook reaches its peak between 1 and 2 days after high water on the main river has passed, while the Concord River peak comes from 2 to 3 days later. Lake Winnepesaukee (drainage area, 360 square miles), Squam Lake (drainage area, 57 square miles) and Newfound Lake (drainage area, 94 square miles) are all big enough to largely control their watersheds.

Flood coefficient of Merrimack River at Franklin Junction, 550-600;* at Garvins Falls, 450-550;* at Lowell, 400-500.*

Merrimack River Tributaries

Pemigewasset River (615 square miles). — Flood coefficient at Plymouth, 600-700. Watershed above Plymouth fan-shaped, steep slopes, not much channel storage. High rainfall.

Suncook River (258 square miles). — Flood coefficient at Suncook, 150-200. Several ponds and reservoirs. Slope of river flat between concentrated falls.

Contoocook River (773 square miles). — Flood coefficient at Penacook, 200-250. Main part of watershed long and narrow, draining northward, but joined near its mouth by branches from the northwest, so that it has an area larger than any other branch of the Merrimack except the Pemigewasset. Fall steeper than in other tributaries, and mostly concentrated between long stretches of flat slopes.

Souhegan River (168 square miles). — Flood coefficient at Merrimac, N. H., 200-300. Moderate slopes, several small ponds, and fall mostly concentrated.

Nashua River (530 square miles). — Flood coefficient at Nashua,

* Based on uncontrolled drainage area.

150–200. A northward drainage basin like the Contoocook. Curious one-sided drainage pattern, with all branches on the west side. The South Branch (118 square miles) controlled by Wachusett Reservoir. North Branch (127 square miles) fairly steep, broken up by a succession of dams, fairly rapid rate of run-off. Slope of main river very flat with broad meadows affording much channel pondage.

Concord River (375 square miles).—Flood coefficient at Lowell, 60–80, due to Concord, Mass., meadows extending 25 miles above North Billerica.

Flood Control Requirements

From 4 to 6 billion cubic feet of flood storage on the 615 square miles of the Pemigewasset above Plymouth would substantially reduce flood peaks on the Merrimack River. Storage for power purposes on the Contoocook, Suncook and small tributary streams, with provision for excess storage above spillways, would materially reduce any local hazard on these streams, and probably improve to some extent conditions on the main river. A suggested diversion of a portion of the upper Pemigewasset River through Lake Winnepesaukee, while very expensive, is no more unusual than many projects already carried out on the Pacific coast.

ANDROSCOGGIN RIVER

The Androscoggin River * (3,470 square miles) (see Fig. 23), with a length of little over 200 miles, drains the region north and east of the Presidential Range (Mt. Washington, El. 6,290), and the central part of the basin is spotted with mountain peaks varying between 3,500 and 4,300 feet in elevation. (See Fig. 23.)

The flow above Errol Dam (1,095 square miles) is almost fully controlled by the 29.5 billion cubic feet of storage in the Rangeley system and the Aziscohos Reservoir. Of the total drainage area of 285 square miles between Errol and Berlin, approximately 123 square miles may be rated as "quick run-off area," or little less than the White Mountain area near Shelburne. The fall between Errol Dam (El. 1,244) and the upper dam at Berlin is about 150 feet, mostly concentrated just below Errol and in the rapids near Pontoocook, with long stretches of rather flat slopes and intervalles in between producing somewhat sluggish characteristics. Below the developed falls at Berlin and Gorham the river runs a distance of 45 miles with an average fall of 3 to 4 feet per mile to Rumford Falls. (Upper Dam El. 600.) From Rumford Falls

* See Report on "Maine Rivers and Their Protection from Possible Flood Hazards," Maine Development Commission, 1929.

to Brunswick, a distance of 83 miles, the greater part of the fall is developed at 14 dams with mill ponds of moderate length, and for the most part with slopes of not over 1 to 2 feet per mile between the successive developments.

Between Gorham and Rumford Falls the Androscoggin is fed by numerous short, quick run-off tributaries from precipitous mountain slopes. With storms of less than 15 to 20 hours' duration the peak on the main river between these points is caused by these tributaries and



FIG. 23. — ANDROSCOGGIN RIVER DRAINAGE AREA

those directly above Berlin, and occurs some 12 hours before the peak arrives from the upper part of the drainage area between Berlin and Errol. In the 1927 flood the peak occurred at Berlin at 8 P.M. on November 4, while the much larger peak at Leadmine Bridge, in Shelburne, some 8 miles lower down, occurred 10 hours earlier, at 10 A.M. on the same day. A considerable retarding effect is caused by channel restrictions at Gilead and immediately above Rumford, producing large volumes of channel pondage. The peak reached Rumford Falls at 4 A.M., November 5, Livermore Falls at 4 P.M., November 5, Gulf Island

at 7 P.M. and Lewiston at 10 P.M. High water occurred at Brunswick early in the morning of November 6. Flood coefficient of main river below Gorham, 400-500.*

The tributaries below Rumford Falls, including the Dead, Nezin-scot, Little Androscoggin and Sabbatus Rivers, all have several lakes and ponds, flat slopes and sluggish flood characteristics.

Effective flood storage on the Androscoggin should be located on the main river above Rumford Falls, as reservoirs on the short, steep tributaries between there and Gorham would hardly be feasible. A large reservoir on the main river above Rumford Falls would materially reduce flood peaks lower down, but probably is not warranted from an economic standpoint at the present time, considering the amount of flood damage that has occurred on this river.

KENNEBEC RIVER

The Kennebec River † (5,970 square miles) extends from the Province of Quebec across the central part of Maine to the Atlantic Ocean at Merrymeeting Bay, a distance of something over 150 miles. (See Fig. 24.) There are numerous mountain peaks throughout the upper parts of the basin at elevations between 2,000 and 3,300 with a maximum elevation of 3,600 for Mt. Bigelow.

The river is well controlled above Moosehead Lake (1,240 square miles), with its sluggish tributaries and a total volume of storage in the system of nearly 35 billion cubic feet. Indian Pond occupies a 5-mile flat stretch of river, with the upper end about 5 miles below the outlet of Moosehead. From the lower end of Indian Pond to Bingham, a distance of 36 miles, the river has steep slopes of 10 to 20 feet per mile. The mill pond above the new development at Bingham will flood back from 8 to 10 miles of this part of the river. From Bingham to the upper end of the Madison mill pond, near North Anson, the slope runs from 4 to 5 feet per mile, with one dam at Solon. Below Madison (El. 244) the fall is largely concentrated at 8 dams.

The banks of the river are generally high, which, with the steep slopes, afford little channel pondage and produce high velocities with quick run-off characteristics. In the 1901 flood the velocities in many places ranged from 10 to 15 feet per second.

The Kennebec River has very high flood characteristics. The peak from the lower part of the Dead River drainage area is somewhat greater and occurs a little earlier than the peak coming down the Kennebec

* Based on uncontrolled drainage area.

† See Report on "Maine Rivers and Their Protection from Possible Flood Hazards," Maine Development Commission, 1929.

from the area below Moosehead Lake and Indian Pond. The peak formed by these two branches at the Forks synchronizes closely with the peaks from the Carrabassett and Sandy Rivers, producing a short, sharp flood wave from Skowhegan down. In the 1901 flood the rise at



FIG. 24. — KENNEBEC RIVER DRAINAGE AREA

Madison (3,200 square miles) began at 8 A.M. on December 15, the peak was reached by midnight of that day, and the river was well on its way down by 10 A.M. on the following morning. At Waterville (4,270 square miles), the peak occurred about 9 A.M. on December 16, and the flow was down to 25 per cent of the peak by 5 P.M. on December 17. Flood coefficient (before construction of Bingham Dam), 800–1,000.

Kennebec River Tributaries

Dead River (878 square miles) enters the Kennebec at the Forks. The upper 500 square miles of this watershed is flat, with much channel pondage and sluggish flood characteristics. The lower part of the basin is steeper with moderately rapid run-off.

Carrabassett River (395 square miles) has steep slopes, little channel pondage and a quick run-off.

Sandy River (670 square miles) has flatter slopes than the Carrabassett, and not quite as rapid run-off.

The remaining tributaries which enter below Skowhegan, including the Sebasticook (970 square miles), the Messalonskee (210 square miles), and the Cobbosseecontee (240 square miles), all have numerous lakes, flat slopes and very sluggish run-off characteristics.

Flood Control Requirements

The influence of the Bingham development, with its long mill pond, will probably be to lower somewhat the flood characteristics below that point. Additional flood protection could best be gained by the further development of the pondage on the main river between North Anson and the Forks, and by the creation of substantial flood storage on the Carrabassett. More storage on the Dead River would also help.

PENOBSCOT RIVER

The Penobscot * drainage area (8,570 square miles) lies to the east of the Kennebec watershed, with a total length of about 160 miles from the headwaters of the West Branch bordering on the Province of Quebec to the Atlantic Ocean at Penobscot Bay (see Fig. 25). The altitude of the basin is lower than that of the Kennebec and Androscoggin, although Mt. Katahdin (El. 5,273), the highest point in the State of Maine, is located between the East and West Branches.

The main river is formed by the junction of its East and West Branches at Medway. From there to tidewater, a distance of 74 miles, the river has a fall of 240 feet, about 90 feet of which is developed at 6 dams below West Enfield. The general slope of the main river between the dams is from 1 to 3 feet per mile.

The drainage basin of the Penobscot River is characterized by its width and compactness, which is in part compensated by the large

* See Report on "Maine Rivers and Their Protection from Possible Flood Hazards," Maine Development Commission, 1929.

number of natural lakes and reservoirs and the flat slopes. None of the tributaries have particularly high flood characteristics, and, due to their differences in slope and channel pondage, do not synchronize exactly with the main river peaks. The main river peaks have their origin with the flood flows from the East Branch, and are increased but



FIG. 25. — PENOBSCOT RIVER DRAINAGE AREA

little by the Mattawamkeag and Passadumkeag, but are chiefly augmented by the flood flows from the Piscataquis and the smaller tributaries. The flood coefficient of the main river, 400–500. The most effective flood storage should be located on the East Branch and the Piscataquis.

Penobscot River Tributaries

The West Branch (2,100 square miles) is well controlled by the 45 billion cubic feet of storage in the Twin Lake system.

The East Branch (1,130 square miles) has an average slope of about 8 feet per mile in the 47 miles from Medway to Grand Lake. There is considerable lake area above Grand Lake. While not controlling the flood run-off, it retards it to a large extent. The run-off from the 634 square miles below Grand Lake is moderately rapid.

Mattawamkeag River (1,500 square miles) enters the Penobscot 11 miles below Medway. The Mattawamkeag River has flat slopes with long stretches of dead water and low banks, resulting in a large volume of natural pondage and sluggish flood characteristics. The peak of Mattawamkeag at its mouth comes about 12 to 20 hours after the peak of the East Branch at Medway, and about the same time as the peak of the main river at West Enfield, some 26 miles below Mattawamkeag.

Piscataquis River (1,500 square miles), with its rapid run-off, enters the Penobscot at West Enfield. Sebec River, the middle of its three main branches, has its flow somewhat retarded by Sebec Lake. Pleasant River and Piscataquis itself have moderately steep slopes without much channel pondage. The peak from the Piscataquis slightly precedes that from up river, and constitutes a substantial part of the flood flow below West Enfield.

Passadumkeag Stream (383 square miles), with its flat slopes, numerous lakes and large volume of channel pondage, is even more sluggish than the Mattawamkeag, and its peaks arrive at the main river from 12 to 24 hours after the main river peak has occurred.

Kenduskeag Stream (214 square miles) has moderate slopes and a moderate rate of run-off, with the peaks preceding those of the main river at Bangor.

Chapter VII. — Flood Prevention Measures

IN GENERAL

Measures for preventing or lessening flood damage range from the construction of extensive works for that specific purpose to a regulation of the proper character of construction of any structure along rivers, as well as provisions for warnings in time of threatened flood. This chapter deals with this subject in the light of the 1927 flood in New England.

Flood prevention works take the form of reservoirs used wholly or in part for flood prevention, detention reservoirs, levees, and channel improvements.

STORAGE RESERVOIRS

Benefits from Reservoirs in November, 1927, Flood

The returns from a questionnaire have been tabulated in Table 29. In many instances reservoirs had a reserve capacity during the flood sufficient to decrease materially the flow at the outlet, and consequently



PLATE VII. — BRIDGE ON MISSIQUOI RIVER, ENOSBURG FALLS, VT.,
NOVEMBER, 1927

the flood damage. There is no assurance, however, that a similar severe flood may not occur at some time when the reservoirs are generally well filled prior to the flood, as flood control is incidental to storage for power and other purposes in New England reservoirs.

Methods of Use and Operation of Power Reservoirs

In the common method of operating reservoirs they will be full or nearly so by the end of the period of spring run-off. They will then be gradually lowered in level, — except as unusual rains occur until the minimum level is reached, — usually in February or March of the next season.

The minimum level is fixed by the consideration of both quantity of storage and permissible variation in head in cases where a power plant is located at the reservoir.

Except for a period in the late spring and early summer, therefore, the water level in power reservoirs will normally be below spillway level, and they will have more or less capacity to absorb floods. Even if filled, when excessive rainfall occurs, storage above spillway level is still available to cut down flood peaks.

By allocating some of the top portions of a reservoir capacity to flood use, a still further measure of flood protection can be obtained.

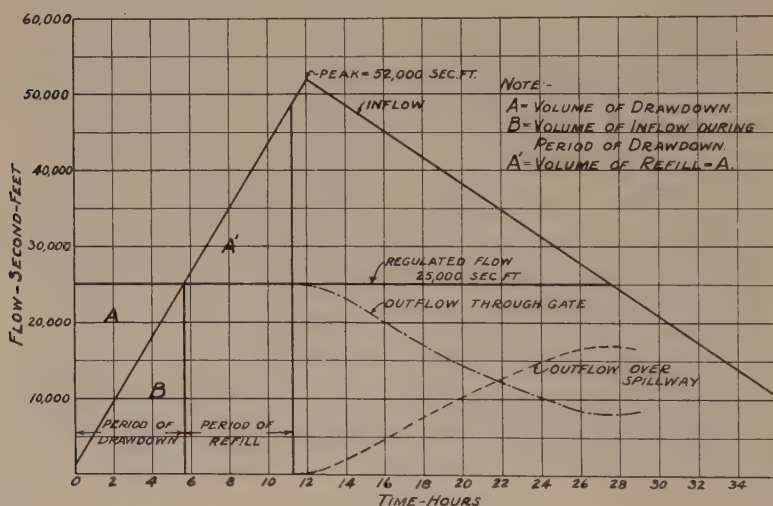


FIG. 26. — EFFECT OF GATE OPERATION AND STORAGE ABOVE SPILLWAY UPON FLOOD FLOW AT PROPOSED GAYSVILLE RESERVOIR SITE ON THE WHITE RIVER

This reservation is unnecessary except during those months when floods may be expected. At other times reserve capacity can be safely used with intelligent operation.

Value of Reservoir for Flood Protection

All reservoirs are to some extent valuable for flood reduction. As discharge occurs over the spillway of a reservoir, the temporary storage of a portion of the flood causes a reduction in the flood peak, and permits more time for flood warnings.

A study has been made of the effect of storage above spillway level upon flood flow at a suggested Gaysville reservoir site on the White River, as shown on Fig. 26, where the curve of inflow is shown with a peak of 52,000 second feet.*

Assuming flood gate capacity equal to 25,000 second feet, the gates should be opened to discharge this amount at the beginning of the flood period. Practically, this could be insured by requiring that, with reservoir full, if a rainfall of, say, 2 inches or more occurred in 6 hours, the gates should be opened to discharge 25,000 second feet. By this time the effect of the rainfall would just about begin to increase the flow at Gaysville, or, in other words, the flood (or possible flood) flow would begin. If rain continued at this rate the gates would be left open, if it ceased, the gates could be shut. If rain continued and a great storm occurred, the method of handling it would be as shown on Fig. 26.

With gates open, a volume of water shown by area "A" would be drawn from the reservoir and the water would be below the spillway level for over 11 hours. At the end of 5.7 hours the inflow would be 25,000 second feet; thereafter, the reservoir water level would gradually rise and reach spillway level at about 11 hours from the beginning of the flood period.

Water would then begin to flow over the spillway, as shown, and the floodgates would be closed gradually from time to time, so as to keep the total flow at about 25,000 second feet.

Such storage *above spillway level* with suitable outlet gate capacity, properly operated, will thus materially reduce flood flow.

DETENTION RESERVOIRS

Detention reservoirs or retarding basins differ from storage reservoirs in that they come into use only at infrequent intervals, and then for only a short time. Water is not held in them in the sense of storage; it is merely delayed or retarded there in its transit downstream.

A detention reservoir is formed by a dam across a valley, with outlets through the base of the dam to permit the ordinary river flow to pass through unobstructed. The size of the outlets are proportioned so that at times of highest floods only such amounts of water will escape through them as can be safely taken care of in the river channels below the dam. The excess water is held back by the dam, accumulating temporarily in the reservoirs, to flow off later through the outlets as

* H. K. Barrows, Report of Consulting Engineer to Advisory Committee of Engineers on Flood Control, Vermont, December, 1928.

TABLE 29. — EFFECT OF RESERVOIR

RIVER AND RESERVOIR	Drainage Area in Square Miles	BELOW SPILL			
		FULL RESERVOIR CAPACITY			Available Storage, November 3 (Inches on Drainage Area)
		Million Cubic Feet	Corresponding Draw-down in Feet	Inches of Depth Over Drainage Area	
	1	2	3	4	5
<i>Maine</i>					
Rapid River:					
Middle Dam Reservoir (Middle Dam, Me.)	104	5,866	17.3	24.3	11
Upper Dam Reservoir	405	8,887	12.2	9.4	2.6
Androscoggin River:					
Androscoggin Dam Reservoir (Wilson Mills, Me.)	233	9,593	45.0	17.7	9.6
Androscoggin River:					
Rangeley Lake (Oquossoc, Me.)	90	992	4	4.7	0.0
<i>New Hampshire</i>					
Newfound River:					
Newfound Lake (8 square miles area)	91	1,200	6	5.7	3.1
Winnepesaukee River:					
Winnepesaukee Lake (71.8 square miles area)	360	7,000	3.7	8.4	6.0
Merrymeeting River:					
Merrymeeting Lake	12.55†	774	16	26.5	3.1
Androscoggin River:					
Errol Dam Reservoir (Errol, N. H.)	353	3,245	9.5	4.0	1.5
Connecticut River:					
First and Second Lakes	83	3,865	26 & 12	20.0	7.74
Mascoma River:					
Goose Pond	15.0	480	20.0	13.8	8.25
Grafton Pond	3.2	139	16.0	18.7	18.0
Crystal Lake	10.5	140	9.0	5.7	1.68
Mascoma Lake	152.0	500	8.5	1.42	0.23
<i>Vermont</i>					
East Creek:					
Chittenden	17	713.7	33.0	18.0	4.87
Deerfield River:					
Somerset	30.0	2,714	85.0	38.9	11.72
Davis Bridge	154*	5,036	90.0	14.1	9.21
<i>Massachusetts</i>					
South Branch-Nashua River:					
Wachusett Reservoir	108.8	7,100	60	28.0	14.8
Whitehall Brook:					
Whitehall Reservoir	4.35	168	10	16.6	0.0
Indian Brook:					
Hopkinton Reservoir	5.86	172	30	12.6	0.0
Stoney Brook:					
Sudbury Reservoir	22.28	270	5	5.2	1.4
Cold Spring Brook:					
Ashland Reservoir	6.43	156	30	10.4	0.0
Sudbury River:					
Framingham No. 1	74.66	18	4	0.10	—
Framingham No. 2	45.14	62	14	0.6	—
Framingham No. 3	27.68	137	20	2.1	0.06
Snake Brook:					
Cochituate	17.58	135	5	3.3	0.0
Westfield Little River:					
Borden Brook Reservoir, Blandford, Mass. (Springfield Water Works)	7.95	334	70	18.1	0.3
<i>Rhode Island</i>					
Pawtuxet River:					
Scituate Reservoir	93	4,900	71	22.6	5.2

* Net.

† Two feet additional freeboard available.

STORAGE ON FLOOD OF NOVEMBER, 1927

WAY CREST				Maximum 24-Hour Flow, Second- Feet, Square Mile	APPROXIMATE CAPACITY ABOVE CREST			Total Storage Available Nov. 3 (Inches on Drain- age Area)	REMAINING Nov. 5	
STORED INCHES ON DRAINAGE AREA		CAPACITY REMAIN- ING NOV. 5			For Depth in Feet of	CORRESPONDING			Inches on Drain- age Area	Per Cent of Total
Maximum Day	Two Days	Inches on Drain- age Area	Per Cent			Inches on Drain- age Area	Second- Feet, Square Mile, 24 Hours			
6	7	8	9	10	11	12	13	14	15	16
2.5 0.6	2.6 1.0	9.1 1.6	37 17	67 16	— 0.5	— 0.37	— 10	— 3.0	— 2.0	— 20
1.0	1.1	8.5	48	27	5.0	3.7	99	13.3	12.2	57
0	0	0.0	0	—	1.0	1.3	35	—	—	—
0.13	3.1	0.0	0.0	3.6	2	2.1	57	5.2	1.0	13
0.04	1.4	4.6	55	1	0.5+	1.2+	32	7.2	5.8	60
—	0.9	2.2	8	—	2	3.5	94	6.6	5.7	19
0.9	1.1	0.4	10	23	1.0	0.4	10.8	1.9	0.8	18
1.79	2.36	5.38	26.5	48	2.0	2.23	60	10.0	7.6	34
.66	1.18	7.07	51.0	17.7	3.0	2.57	70.0	10.8	9.6	59
2.83	2.83	15.17	81.0	76.0	2.0	4.0	107.0	22.0	19.2	85
1.89	2.10	0	0	51.0	2.0	1.6	43.0	3.3	1.2	16
.58	0.93	0	0	15.6	3.0	0.73	20.0	1.0	0.0	0
4.05	4.87	0.0	0.0	107.0	4.0	3.4	91.0	8.3	3.4	16.0
4.78	5.44	6.28	16.2	129.0	4.0	4.35	117.0	16.1	10.6	25.0
4.53	5.2	4.01	28.5	121.0	5.0	1.36	37.0	10.6	5.4	35.0
—	1.3	13.5	48	—	1.5	1.1	29.0	15.9	14.6	50
0	0	0.0	0.0	—	2.0	5.2	141	5.0	2.1	9.6
0	0	0.0	0.0	—	1	0.6	16	0.0	0.0	0.0
1.3	1.4	0.0	0.0	35	1	1.1	29.1	2.5	0.1	1.6
0	0	0.0	0	—	1.0	0.5	13.4	0.0	0.0	0
—	—	0.7	64	—	3.2	0.12	3.2	—	0.19	83
—	—	0.11	19	—	2	0.11	3.1	—	0.17	24.0
0.06	0.06	0.0	0.0	1.6	1.5	0.25	6.70	0.31	0.06	2.5
—	—	0.0	0.0	—	2.3	1.8	48.6	0.3	0.0	0
0.3	0.3	0.0	0.0	—	6.0	0.37	10	0.67	0.15†	0.8†
1.9	3.0	2.2	10	51	5.5	3.9	105	9.1	6.1	23

Note. — Col. 10 = $\frac{\text{Col. 6}}{0.0372}$

the flood subsides. Obviously, a detention reservoir is of no value for power purposes, and its justification must depend solely on its flood-reducing tendencies.

The most comprehensive system of detention reservoirs constructed to date is that of the Miami Conservancy District, on the Miami River and its tributaries in Ohio, built in 1917-23, as a result of the disastrous floods of March, 1913. The flood district has constructed five large detention reservoirs with a total capacity above Hamilton of 36,680 m. c. f. or 10.5 m. c. f. per square mile, equivalent to 4.5 inches depth on the drainage area. These reservoirs were designed for a run-off of 11.5 inches in 4 days, or about 40 per cent more than that which occurred in the storm of 1913.

The flood works on the Arkansas River at Pueblo, Col., completed in 1924, consist of a detention reservoir on the river above Pueblo at Rock Canyon, with a capacity of about 1,050 m. c. f. for a drainage area of 4,800 square miles, combined with a new river channel through the city of Pueblo of approximately 100,000 second feet capacity. This Pueblo detention reservoir will give a regulated flow of about $\frac{5}{8}$ of the unregulated peak — a considerably less degree of regulation than in the Miami valley, as in this case channel improvement constitutes the major factor in flood protection.

The valley land of a detention reservoir is submerged but a very small per cent of the time, often only at intervals of some years. It may, therefore, be kept in use for such purposes as farming, subject to occasional flooding, and therefore some revenue may be obtained from it.

It is doubtful if detention reservoirs are practical for flood control in New England, owing to the extensive use of the river valleys by railroads, highways and settlements, and the consequent high cost of land required.

CHANNEL IMPROVEMENTS

The field of channel improvements in New England is very limited because the topography of the river valleys generally permitted cities and towns to be located on terraces above the river beds rather than on the flat areas immediately bordering them. The freshet of November, 1927, with all its destruction and loss of life in Vermont, and, to a lesser degree elsewhere, has not yet resulted in any demand for channel improvements as the permanent remedy.

While many meadows and low areas in New England have been regularly flooded, and cellars of mills and houses near the streams as regularly been wet from rising rivers or tributary streams, it has never

been found necessary to regulate a stream of any material size; and it is only in the congested districts of the Middle West and Mississippi Valley, and a few cities or districts badly situated, that regulation has been necessary. Perhaps future floods coming when cities and towns of New England are more congested may require channel improvements in some places.

During the freshet of November, 1927, the rapid run-off from tributary streams caused the tremendous rise from normal to flood heights, in some cases reaching a peak of 30 to 50 feet. This situation precludes any control of these tributary streams by any form of channel improvements.

While the main rivers of New England — the Connecticut, Merrimack and some of the rivers of Maine — rose to a height of 15 to 30 feet above low water, and while they overflowed the adjacent lowlands and meadows, no serious damage to life or property resulted.

Removal of Obstructions to Flow

The removal of obstructions may be worth while in certain localities. Trees, brush, sunken logs, boulders, ledges, sand bars, all help to obstruct channels, and their removal is often desirable; but instances of this kind of channel improvement are rare. Each freshet has left its trail behind, and nothing has been moved until the next one came along.

Most of the work of this character in New England has been the improvements by the log-driving companies of the channels of the larger rivers and tributary streams in the days of log driving, among which those of the Kennebec and upper Connecticut are best known.

The United States government, for the purpose of improving navigation, has dredged harbors and done some work on the channels of the Connecticut, Merrimack and rivers of Maine, but its influence on floods is relatively small.

Dams and flashboards, inadequate culverts or bridge openings may be serious obstructions to the flow of streams, and offer one of the most difficult problems in the way of permanent relief from flood damage. Bridge spans have often been shortened by borrowing from the river bed. Many of the Fitchburg Railroad bridges were destroyed by the Hoosac River surrounding and cutting around the abutments, leaving them high and dry. In one case, Bridge No. 233 was put in commission by using one abutment for a central pier and building another span over the new channel of the river.

Dams are usually the sites of manufacturing plants which may be

the life blood of the town, and are too important to remove. But they should be located and built with care as to their effect upon flood heights. A dam in the town of Maynard, Mass., across the Assabet River, has been taken down as a part of the rebuilding of a new bridge since the destruction of the old bridge by the November, 1927, freshet. After the Westfield River freshet, of December 10, 1878, the spillway of the Horton Dam was lengthened.

In many cases reservoir dams have been overtopped and gone out because of the spillway having become clogged with brush, logs or other débris which had accumulated through long periods of normal



PLATE VIII. — INTERIOR OF ESSEX JUNCTION POWER STATION,
WINOOSKI RIVER, NOVEMBER, 1927

High-water mark visible on penstocks and wall about 2 feet above clock.

No. 4 unit ran throughout the flood and can be seen in operation

or low water. The general care and supervision of dams and reservoirs by some county or state body should include the removal of such obstructions.

Improving Channel

For relatively small drainage areas in the congested city and metropolitan districts of New England, channel improvements are common enough, but chiefly in connection with the sewerage and drainage of our cities and large towns. The improvement of Stony Brook, Boston, and the drainage systems of Worcester, Springfield, and other cities

and towns, illustrate the necessity of channel improvements on a small scale.

A well-known example is that of Stony Brook,* mentioned before, which, after a series of freshets overflowed its banks and flooded homes and manufacturing property, was first straightened and the obstructions removed, and later the flow confined to a smooth masonry channel.



PLATE IX. — GORGE CUT BY BLACK RIVER AT CAVENDISH, VT.,
NOVEMBER, 1927

Levees

The best examples of levees in New England are those of Westfield, Mass., to control the flood waters of the river of that name, and those at West Springfield on the west bank of the Connecticut River. The levees at Westfield had existed for many years, but were rebuilt as a result of the freshet of December 10, 1878.†

From a study of the best authorities and the records of floods on rivers, the banks of which are protected by levees, it is assumed that,

* "Prevention of Floods in the Valley of Stony Brook," Report of a Commission, James B. Francis, Eliot C. Clarke, Clemens Herschel, February 15, 1886.

† Hiram F. Mills, "Best Plan for the Permanent Protection of the Town from Future Floods," Westfield, 1879.

except in rare cases, it is neither practicable nor wise to levee the upper or tributary branches of our New England rivers with their steep slopes. Farther down the streams on the main rivers, it is only locally where some valuable farming or building sites require diking, or isolated cases like Westfield, where certain portions of the town were laid out within the flooded areas, that levees will be built.

In many of our cities, like Concord and Manchester, N. H., and Lowell, Mass., the presence of low but cheap land has brought into the market unsuitable areas which have been built upon between the warnings given by the occasional freshets. The cost of protection for these lands is so high that they should be left exposed to the occasional freshet, provided that there is not the danger of loss of life which is usually not present because of the many hours of warning.

The whole problem is somewhat complicated, but as in every other remedy to be studied, insurance and saving of life and property should be weighed against the chance of recurrence and the investment.

There is evidently little field for the construction of levees and dikes in New England; but rather, floods should be avoided by building on the second terrace, leaving the rivers free to rise and fall in the banks and channels which they have cut for themselves. It will be many years before the increase of population in New England and shortage of land for building, farming and other purposes will require the protection of bottom lands by levees.

RIVER STRUCTURES

The previous discussion relates to works constructed or used for the purpose of flood protection. Much may be done in lessening flood heights and damage by the proper design and construction of structures of all kinds along rivers, and by provisions for regulating such construction.

The damage done during the 1927 and previous large floods was in many cases aggravated by local factors, and could have been lessened or prevented. A brief study of these local factors is necessary before considering proper design and construction of river structures which follows. Local factors may well be termed those factors or conditions which so affect or influence a moderate sized flood as to cause local damage, or increase the local effects of a really large flood. Outstanding factors of this nature are: failures of dams, channel obstructions, effects of logging operations, and ice jams.

Failures of Dams

Since the Mill River calamity of 1874, New England has been remarkably free from great disasters caused by the failure of dams. There are a number of reasons why this should be so. There are not many large dams; the foundation conditions in general are not complicated; state supervision has been in force for some time in one form or another in most of the New England States; and such large projects as have been carried out have been under the direction and supervision of the best engineering talent.

Of the failures which have occurred among the smaller New England dams, that of Becket Dam in western Massachusetts, at the time of the November, 1927, flood, was probably one of the most disastrous, causing loss of life and great damage to the Boston & Albany Railroad, highways and private property. This was an old earth dam about 20 feet high, with a 3-foot wood stave outlet pipe. A spillway was located at some distance from the dam. The failure is reported to have occurred before the pond filled to the spillway level, and evidently started in the vicinity of the wooden pipe, which apparently had no water cut-offs along its line. Although the reservoir controlled by this dam covered less than 100 acres with an average depth of 5 to 10 feet, the suddenness of the release of the water, combined with an already swollen brook and narrow valley below, was such that great damage was done.

The failure of the Glen Dam, near Rutland, Vt., during the same flood had little effect upon the damage along the stream below, as the dam was only a small diverting dam, and the failure occurred before the peak of the flood was reached. In this case the water topped the approach embankment.

In July, 1915, a 4-inch rainfall in southern Vermont caused a flood of 125 second feet per square mile on the Deerfield River at Readsboro, Vt. At this point there existed a 40-foot timber crib dam, built in 1888. When the water reached a height of 8.5 feet above the crest, it began to run through the timber crib abutment on the right bank, and quickly cut a channel 50 feet wide. No other damage was done, however, as the pond had an area of only 26 acres. At a dam 5 miles below, the rise of water due to the break appeared to be about 3 feet. The spillway section of the Readsboro Dam was located on ledge and stood intact, showing no ill effects from the washout of the abutment.

A timber crib dam across the Connecticut River at Gilman, Vt., went out on April 10, 1928. This dam was about 25 feet high, and was

30 years old. This failure occurred with a near maximum flood, combined with a heavy ice flow. The initial failure was caused by the cakes of field ice battering to pieces the timber abutment, after which the ice and water unraveled a large part of the spillway section. The abutment and spillway were located on earth. The failure was due to the deteriorated condition of the dam above the normal tailwater level. No damage occurred from this failure beyond the loss of the dam itself. The effect on the crest of the flood at Waterford Bridge 10 miles below was about 2 feet.

The failure of the Abenaki Dam, near the Dixville Notch in northern New Hampshire, on May 3, 1929, caused local damage, but fortunately no loss of life. This failure is of particular interest because of the small amount of water involved in relation to the amount of damage done.

The drainage area at the point of failure probably does not exceed 5 square miles. The water held back by the dam was something like 500 acre feet, yet the damage to highways, bridges, buildings and lands has been estimated from \$200,000 to \$500,000.

The dam was an earth structure with a concrete core wall, which had been in use over 20 years. The spillway was in the nature of a concrete box culvert 2 feet high, 16 feet wide, and extending 30 to 40 feet across and under the state highway which ran the length of the dam. The pond was divided into two nearly equal parts by a highway embankment, with a 6-foot circular concrete culvert connecting the two parts.

Fortunately the failure occurred in the late afternoon and was discovered in the early stages, which permitted a warning to be given to the people below.

The report of the New Hampshire Public Service Commission gives the cause of failure as the breaking of the floor of the concrete culvert near the center of the top of the embankment due to settlement or frost action, and the resulting admission of water to the embankment downstream of the core wall which found its way along the underside of the spillway down the slope. At the time of the failure water was flowing through the culvert about 1 foot deep.

From the time the dam gave way until the adjacent part of the pond was empty was just 10 minutes. A few minutes later the highway dike gave way, and the part of the pond behind the highway dike emptied in 10 minutes more. This gave two 10-minute discharges of an average of about 18,000 second feet. The water reached Colebrook, 10 miles below, in 1 hour, which was at the average velocity of 15 feet per second. So rapid was the rise and fall of the water in Colebrook that, whereas

the water surrounded the Colebrook Hotel to a height of 4 feet above the first floor, yet no water came above the first floor on the inside of the building.

The valleys of the smaller streams, with their improvements and inhabitants immediately below a failure, are the great sufferers from failure of dams, while such failures really have a very minor effect on the extreme floods and flood damage of our larger streams. More is printed about one dam that fails than about a thousand which stand firm. In fact, under the stress of flood time excitement injustice is often done to perfectly good dams by false reports of their failure. However, even such false publicity tends to emphasize the importance of safety, and in this respect may serve some useful purpose.

It is a notable fact that the 1927 flood in Vermont was nowhere due to the release of large volumes of water by the failure of dams. No dams of large size or that stored large quantities of water went out, and most of the failures that did occur were due to washing out around the ends, usually a somewhat gradual process. In every valley in which large reservoirs existed these reservoirs operated to reduce flood flow and flood damage. It is true, however, that in some cases the existence of old dams that did not fail increased the damage done. For instance, if there had been no dam at Bolton Falls, 4 miles down the Winooski River from Waterbury, the water would not have reached so high an elevation in that village, and the Central Vermont Railway would probably have been spared the worst washout on its line. Also at Cavendish, on the Black River, a washout said to have been the greatest in the state, and to have carried away half a million cubic yards of earth as well as a part of the village, might not have occurred if there had been no dam at the head of the Cavendish Gorge.

State supervision of dams and reservoirs is an effective check on safe construction. An equally important function of state supervision should be a continuing system of checking the condition and methods of operation of dams, the failures of which might be disastrous. Disuse may lead to neglect, deterioration, and eventually failure. Changes in methods of operation, fixed structures, or spillway crest control mechanism may result in unsafe conditions. State supervision should cover these points, as well as the safety of the structure as first built.

Channel Obstructions

A considerable part of the damage done by the Vermont flood of November 3-4, 1927, was due to the effect of channel obstructions of various kinds. These obstructions were, in many instances, natural,

usually narrow rocky gorges, but a large number were artificial. The latter were of two kinds: permanent structures, such as bridge abutments, piers and buildings, and temporary obstructions caused by masses of floating *débris* lodging against bridges, trees or on shallow spots.

Two examples will serve to show the effects of natural channel obstructions. Two or three miles east of Burlington, the Winooski River flows through what is locally known as the Limekiln Gorge, a vertical walled limestone chasm 75 to 100 feet deep, and in places no more than 50 or 60 feet wide. This gorge is spanned by a concrete arch



PLATE X. — LIMEKILN GORGE ON THE WINOOSKI
RIVER ABOVE BURLINGTON, VT.

bridge with floor 75 feet above the river, with a normal depth of water under it of about 17 feet. Above the gorge the valley opens out into a broad fertile intervalle of perhaps 1,500 acres. At the crest of the flood, with a discharge of somewhat over 100,000 second feet, the water just above the gorge was raised about 50 feet above the normal level, turning the intervalle into a muddy lake, drowning many head of livestock and flooding the generator room of the Essex Junction Power Plant, 3 miles upstream, to a depth of 20 feet. At no other place was so great a rise above normal water level recorded.

Half a mile below the Limekiln Bridge, where the gorge is somewhat wider, is located an old power plant with a 30-foot concrete gravity

dam. This dam sustained a head of 27 feet of water at the height of the flood, a record in this respect.

The second, and more disastrous example of the effect of a narrow gorge was the great washout at Gaysville on the White River. The valley at this point was about 800 feet wide, with steep wooded hills rising on both sides, and the river flowed in an easterly direction along the south side of the valley, where it had cut a narrow cleft some 40 or 50 feet deep through the ledge. A street of the village ran along the foot of the hills on each side of the valley, and a third ran across the valley, connecting the other two and crossing the gorge on a small steel bridge. The White River Railroad, a small branch line, followed the north bank of the river, and the village with about 30 buildings was located on the opposite side. At the head of the gorge a short distance above the bridge were a low wooden dam and a small hydroelectric plant. The drainage area of the White River above Gaysville is 226 square miles, and the peak discharge could hardly have been less than 250 second feet per square mile, or over 60,000 second feet. This enormous flow overtopped the north wall of the gorge, carried away the bridge, and soon began eating into the glacial drift overlying the rock north of the gorge, undermining and sweeping away the buildings in the village. When the water finally subsided every vestige of the street and the buildings on it had disappeared, and the river was flowing in a new channel on the north side of the valley, on a wide gravel bed at a lower elevation than the bottom of the old gorge. The bedrock, which had been supposed to slope gradually upward from the edge of the gorge to the hills on the north side of the valley, was now seen to slope up for only a short distance from the gorge, and then to dip sharply down to the north below the present bed of the river.

The outstanding example of channel obstruction by structures was along the North Branch of the Winooski River in the city of Montpelier, where buildings had been constructed along both sides of the stream, and five bridges of very limited waterway built across it. While these may not have had a great effect on the maximum flood height in the city, they did result in deflecting powerful currents into the streets, and all but one of the bridges were carried away. One of them, however, a steel pony truss, was later rescued from a prominent citizen's front lawn, and moved back to its former location. It is to be noted that new bridges have been built with no greater waterway than the old ones had, thus inviting trouble in the future unless flood control work is undertaken on this stream.

Examples of damage resulting from channel obstructions could be

cited on nearly every considerable stream in Vermont, but one more case will be mentioned. Waterbury, a prosperous and well kept village of about 2,000 people, lies on the right bank of the Winooski River, 12 miles below Montpelier, most of it along one main street near the foot of the hills that rise steeply from the river flood plain. At the upstream end of the village the highway crossed the river on a heavy steel bridge with a main span of 163 feet and a short approach span. When the water level approached the floor of this structure, the bridge began to catch floating drift, and formed a partial dam that served to turn the greatest force of the current down the main street of the village, destroying several houses near the bridge, with considerable loss of life, and sweeping others off their foundations. Finally, an old "double-track" covered highway bridge floated down from a location about 3 miles above, and forced the longer steel span off its supports, and the subsiding waters disclosed it lying bottom up in the river some 200 feet below its former position.

Near the downstream end of the village, Winooski Street turned off at right angles from Main Street toward the river, one-third of a mile distant, and crossed it by an old wooden covered bridge of about 200 feet span, and of very solid construction. Along this street, as it neared the river, was a row of large elm trees. As soon as the water left the river channel and spread over the flats, these trees began to catch floating drift, and there was built up here in a short time probably the largest pile of wreckage formed by the flood anywhere. In it were thousands of feet of lumber, much of it from sawmills in the Mad River valley, more than 10 miles away, two complete houses, the wreckage from many others and from numerous barns, garages, etc., together with trees and miscellaneous débris of all kinds — thousands of cubic yards in all. This huge accumulation acted as a sort of wing dam to deflect the current toward the lower part of the town, and contributed to the general destruction. The old covered bridge at the foot of Winooski Street floated downstream at the height of the flood, knocked from its piers a new five-span deck plate girder bridge of the Central Vermont Railway a mile below, and finally grounded, not much damaged, in a field beside the main highway, where it was finally taken apart to furnish timber for temporary reconstruction. The plate girder spans were afterward pulled out of the river bed and set back on their piers, and are now in service again.

Numerous other examples of the effects of channel obstructions, both natural and artificial, in causing or increasing damage, might be noted, such as the almost complete flooding of the village of Johnson

by reason of the constriction of the channel of the Lamoille River at a narrow rocky defile 2 miles below, and the destruction of numerous bridges by the pressure of the water against accumulated drift, but these few serve to indicate the part played throughout the state by this particular phase of the flood. These effects were by no means confined to the larger rivers, for many of the small tributaries, some mere trickles under ordinary conditions, brought down almost unbelievable quantities of sand and gravel, and left it piled on highways, spread over once fertile fields, or choking their own channels at points of diminished slope.

In conclusion, it may be said that it is very difficult, if not impossible, when observing the completed destruction, to determine just how much of it should be attributed to each of several causes, of which channel obstruction is only one, and those who witnessed the flood were too busy and excited to note accurately its behavior.

Effects of Logging Operations

One of the earliest uses of our rivers, aside from transportation, was the floating of logs to the mills. So long as rivers remain in their wild state, with no improvement on or adjacent thereto, the log-driving industry is not a factor of particular interest in connection with floods and flood damage, but where improvements are made along the stream, such operations may be of moment.

Direct damages from log-driving operations are most likely to occur at restricted sections of the river, as at bridges, or where the water is shallow. Where jams occur, the water upstream is raised until sufficient pressure moves the jam, or the water builds up sufficient head to force its way through the jam, over the top or to a new location. This excessive backwater has in some cases washed out embankments and roadbeds of highways and railroads which were located at a low elevation along the river, such as on the Connecticut River at North Stratford, N. H., in April, 1914.

In cases where jams have occurred at bridges, the latter have suffered, but due to the better modern construction of bridges and a decreased use of the streams for log-driving purposes, damage from this cause is now fairly infrequent.

Ice Jams and Movements

Of all the uncertain elements which confront the engineer in taking up the problem of a new structure across or adjacent to one of our northern streams, that of ice behavior is the most indefinite. So numer-

ous are the vagaries of this element of the problem that should an attempt be made to guard against every possible source of trouble indicated from observation of past effects in other locations, many projects would be discarded as not feasible.

In our New England lakes and larger ponds the ice forms to considerable thickness, and in the spring melts in place, except as the wind may affect the process. In the natural rivers where a current exists, ice movements occur under a multitude of different conditions. After observation of an unaltered stretch of river over many years one can fairly well catalogue all the various possibilities for such a section of river. If the conditions are changed by building a dam or other structure, observations must be made anew.

Accumulations of frazil ice, sometimes a serious river obstruction, may become many feet deep downstream from a long section of rapid water. Thus, below the Fifteen Mile Falls section of the Connecticut River, great masses of frazil ice are built up for some 6 miles downstream from the foot of the steep rapids. The channel in which the river may be running is clogged with this ice, and the water forced out of the channel to a new location, which in turn is filled with the formation. Thus, the stream is forced from one location to another where the valley is broad, and gradually builds the ice to higher and higher elevations. In this location during the late winter of 1928-29, depths of this ice amounted to 15 or 20 feet for a distance of 6 miles. At the lower end of this 6-mile section, the upper end of the Ryegate Paper Company pond begins, which is 4 miles long. During the ice period, the upper end of this pond ice was gradually forced up to a maximum height of 15 feet, so that the pond ice from the upper end to the mill had a slope of 15 feet, where, under normal conditions, the pond is practically level.

Another form of ice jam is due to ice movements which usually start by the ice breaking up in the more rapid parts of a stream, and moving down against field ice located on quiet water. Such lodging of the ice, which at first may be a single cake, causes obstructions to the flow of the water, and more of the ice sheet is broken up, which in turn moves with the current. This process is repeated until a considerable volume of broken up field ice has been accumulated, which may obstruct the flow of the stream. Such a break-up which occurs in January is likely to be more serious than one late in the season, because of the strength of the ice. On the other hand, ice movements which occur late in the season usually have little difficulty in breaking up the pond ice and moving through the pond with no great damage.

In addition to the overflowing of property which ice jams may cause, they often carry out bridges and other structures adjacent to the stream. The highway bridge across the Connecticut River at Brattleboro, Vt., was taken out by ice in the spring of 1920. It was a 320-foot, single span, steel truss bridge. The jam under the bridge was pushed higher and higher until it raised the bridge from the abutments and carried it downstream.

At Bangor, Me., during the high water in March, 1902, a jam composed of ice and logs lodged against a granite pier of the Maine Central Railroad bridge across the Penobscot River. The pier was sheared off, and the two steel trusses of the railroad bridge, riding on top of the jam, cut the center span out of the covered wooden highway bridge just below. The steel trusses then dropped through the jam and were deposited on the bottom of the river just below the highway bridge. The jam moved about one mile downstream and lodged again. The combined effect of the jam at this location, made up of ice and logs, together with the high tide and flood stage of the river, caused the water in the main river and Kenduskeag Stream to back up enough to flood the streets in the business district of Bangor to a considerable depth.

Railroads have been especially subject to damage from ice movements, not only to their bridges, but to embankments and other sections of roadbed. There is no way of predicting just how an ice jam will move, and it is often the case that a jam moving parallel with a railroad embankment may force the water so high as to flow over the top of the embankment and wash away the roadbed. Another form of damage, common to highway and railroad, may occur where the roadbed is located on a shelf along the river bank. Here the ice may raise the water 20 or 30 feet, and after holding it in this position for a time sufficient to saturate the bank, the jam may give way and the water drop a number of feet within a few minutes, sometimes causing a sliding of the bank.

Ice movements in relatively flat river valleys very often cause a complete stopping of the water in the regular channel, and force the stream across low lying lands, cutting new channels. Piers erected in ponds and river channels for log-driving operations or other purposes have contributed to this type of trouble in some instances.

DESIGN AND LOCATION OF RIVER STRUCTURES

This discussion is limited to certain aspects which were brought out by the experience of the 1927 flood, which has again brought attention to the necessity of providing ample spillway capacity on dams, and the data obtained from it are of great value in estimating flood flows that may be expected.

The amount of flood capacity to be allowed on any dam will depend on many conditions. Important structures on large rivers above settled communities, where failure means loss of life and property, will be designed to take larger floods than smaller structures, where damage due to their failure is small.

The effect of storage above the dam in reducing flood peaks and the spillway capacity required may properly be taken into account in spillway design. Present practice on the larger size earth dams in New England, however, differs in this respect. The Davis Bridge spillway was designed to take 200 cubic feet per second per square mile (50 per cent larger than any previous Deerfield flood), with the storage definitely counted on as an element in providing against greater floods. The new Cobble Mountain Dam, now being built for the city of Springfield, however, has a spillway capacity of 20,000 cubic feet per second, or about 450 cubic feet per second per square mile, regardless of the large storage available to cut down the peak. The new Scituate Reservoir Dam of the Providence water supply allows a spillway capacity of about 400 cubic feet per second per square mile on 92.8 square miles. In these cases, apparently, the storage capacity was not counted on to any great extent in reducing the peak flow for spillway design, except as a factor of safety. Storage above the crest of the dam is an element that may be definitely counted upon in reducing the maximum peak flow over the spillway, the extent to which its full value is allowed depending on the circumstances.

Whether it is safe to count upon the use of waste gates as a part of spillway capacity is another question raised. There were cases in the 1927 flood where small surface gates failed to function as part of the spillway capacity, either because they were not raised, or else were blocked up by trash. It is doubtful whether surface gates under 8 feet in width should be counted upon as spillway capacity. Submerged sluice gates within the dam structure proved themselves satisfactory for flood discharge.

The spillway capacity will determine in part the height of the abutments, and this again will depend upon the importance of the structure.

The abutments of a small dam, the failure of which would not cause damage, may be designed with as low as 1 foot of free board over a flood that is likely to occur once in 100 years, while a high earth dam will be given a large free board over a flood that is considered the maximum for the spillway as designed. There were many abutments overtopped during the 1927 flood, and some were protected by sand bags, others were actually overflowed without doing more than local damage, while still others failed.

Experience showed that an embankment with a core wall built of reinforced concrete, with fill on the downstream side with considerable stone in it, successfully withstood 2 feet of water over it for the short period of the flood, whereas an embankment with sand filling on the downstream side and a core wall without reinforcement failed. Core walls for abutments should be reinforced against water pressure from the upstream side, and, where possible, the fill on the downstream side should be of heavy enough material to resist serious washing. Attention is also called to the possibility of carrying core walls in embankments up above the top of the embankment for maximum flood protection, and the possibility of providing for the use of some kind of flashboards on abutments of minor structures where there are attendants available.

Experience with flashboards during the flood was, on the whole, fairly satisfactory. On the larger rivers the flood warning was sufficiently early so that flashboards could be removed before the flood peak arrived. In most other places the flashboards went off as designed. There were some places, however, where non-automatic flashboards were not tripped in time. Experience indicates that flashboards on dams where constant attendants are not available, or where the size of the stream is such that flood warnings from upriver may not be obtained in ample time, should be made automatic without the necessity of tripping braces or any other human actions.

Many power station floors were submerged during the 1927 flood, and while the damage was not permanent, the interruption of operation and cost of cleaning up was great. With the flood height at the power station predicted, it is clear that there should be as few openings as possible below this level, and those that are necessary should have provision for closing during flood. Seepage and leakage into the portions of power stations below flood water may cause trouble, and in one large station fire engines were used to pump the water out, pointing to the advisability of providing a sump pump in all cases where floors are below maximum high water level. Even in small stations there should

be no difficulty in keeping all the electric apparatus, including that outdoors, above extreme high water, particularly in those with vertical units.

Penstocks along river banks in some cases were badly damaged or destroyed by the high water in the river, indicating the importance of constructing them above flood level or protecting them against the flood.

There were many bridges of all sizes and types which failed in the 1927 flood. The experience of this flood seems to point to the necessity of building bridges higher and with larger openings than has been the past practice. In highway work this does not mean, necessarily, that the approach need be carried to the height of the bridge, but the bridge floor itself should be above the maximum high-water line and with sufficiently large clearance to permit trash and débris to pass underneath during floods. In many cases the expense of building the approaches of highway bridges up to a height above a maximum flood for their full length is prohibitive. In such cases the approaches may be overflowed during maximum high water, but their repair does not usually present as serious a matter as the repair or renewal of the bridge, and the additional flood channel will often allow the bridge to be somewhat lower and yet safer.

Many of the old covered wooden bridges, common on the country roads of New England, came through this flood unharmed. This was due chiefly to their high elevations above the river, based possibly on experience with earlier floods. Though the bridge may be placed well up out of the reach of high water, the approach is often along natural ground which may be overflowed during a flood, thus providing a large flood channel.

Many bridges failed in 1927, due to the water piling up against them, either with or without trash (the latter sometimes consisting of remains of bridges from upstream), and carrying them away. Arch bridges are particularly liable to present serious constriction as the water rises to flood heights, and should be scrutinized from this point of view. A definite part of the Miami Conservancy Flood control plan was the removal of the channel constrictions by the enlargement of bridges. The maximum allowable velocity was 13.2 feet per second, and where velocities in excess of 10 feet per second would exist, special precautions were taken with the piers and river banks. The hydraulic design of a bridge for flood discharge should be given greater attention, for many bridges have insufficient waterways to handle large floods safely.

There were many failures of piers and abutments, due in some cases to the faulty construction, but in many cases to the scour caused

by the high velocity resulting from the constriction of the river channel made by the bridge itself. In one case the original river bed was lowered 15 feet. Velocities of over 10 to 15 feet per second, depending on conditions, are likely to cause more or less serious scour, and any channel with obstructions which causes velocities above this is likely to suffer.

Table 30 gives some measurements of the drop in water surface at bridges and the resulting velocities during the 1927 flood in New England. Another similar record on the Miami River during the 1913 flood showed drops of 1.3 to 4.7 feet, and in one case 8.1 to 8.9 feet, with velocities of 6.11 to 23.1 feet per second.* Constriction of the river channel due to the construction of a bridge may not only cause backing up the water with consequent damage upstream, but also the destruction of the bridge itself due to the water pressure against it. It is not to be expected that bridges can be built without restricting the cross-sectional area of the river for flood discharge somewhat, but many are being built which do this to a dangerous extent.

Another case of damage in the 1927 flood due to constriction of channels was to railroad embankments running along river banks, where the high velocity of flood flow caused washouts. The exceptionally high water reached embankments unprotected against wash, and the unusual velocity in cases affected even protected slopes.

Another element in flood heights is the restriction of the river channel due to buildings, river walls, and other structures built along the banks of rivers in settled communities. It is often the case that this kind of construction is made without adequate hydraulic engineering advice, but where necessary, such construction should be controlled through the establishing of legal restrictions as a part of a complete plan of possible channel improvements.

More attention paid to the design of all structures on or across rivers from the point of view of flood conditions is required. The construction of dams at the present time is usually done with adequate engineering design and supervision. The same care should be applied to other river structures because of their effect upon flood heights.

* Miami Conservancy District Technical Reports, Part IV, p. 59; Part VII, p. 336.

TABLE 30. — FLOOD OF NOVEMBER, 1927, DROP IN WATER SURFACE AT BRIDGES AND RESULTING VELOCITY

LOCATION	Drop (Feet)	Velocity of Approach (Feet)	Maximum Velocity in Section (Feet)	Peak Discharge (Second- Feet)
Wildcat Brook, at Jackson, N. H. .	3.23	12	17.50	4,080
Beebe River, at Beebe River, N. H. .	1.70	8	11.20	5,800
South Branch of Bakers River, at West Rumney, N. H.	1.90	11	14.70	9,800
Mohawk River, at Colebrook, N. H. .	3.30	12	16.86	5,110
Connecticut River, at White River Junction, Vt.	3.00	8*	—	148,000
East Creek, Rutland, Vt.:				
Twin Bridge	2.70	—	—	—
Lester Bridge	1.90	—	—	—
McKinley Avenue	1.00	—	—	—

* Estimated.

RESTRICTION OF BUILDING

The restriction of building in areas subject to flood should provide for the protection of the owner, and the public from river encroachment. The first should be taken care of by providing for the public use of real estate dealers, architects and builders information regarding flood heights to be expected along the river banks. This information should cover both the frequency and magnitude of the flood to be expected; that is, areas should be classified as those likely to be flooded by normal high water, by a flood occurring once in 10 or 20 years, and by the occasional large flood. This information should be on file at some town or city office, preferably in the form of a map, giving the depth of water occurring under the different flood classifications.

For the purpose of the prevention of building in areas where they will cause serious restrictions of river flow, studies may be made of flood flow through cities and towns, and from such studies certain limits in building lines established beyond which no building is to be done. Such lines are common in some of the larger cities, based upon either city ordinance or private agreement. In most of the smaller communities, however, encroachments on rivers subject to floods are in no way restricted.

It is possible that some arrangement for restriction of buildings in flooded areas could be incorporated in the zoning laws. The general

principle of zoning laws is to protect an owner in a given district against improper building by another owner in the district. Zoning against building in areas subject to flood does not come under this general purpose of the zoning laws, it being for the protection of the individual against natural difficulties rather than against the act of his neighbor. From the point of view of river encroachment, however, it is possible that the restrictions which a city or town may consider necessary to prevent undue encroachment on a river channel by building could be exercised through the general zoning law. This assumes, however, that the zoning laws include the right to prohibit as well as regulate buildings to be built, and most of them do not go as far as this.

FLOOD WARNINGS

General

The subject of flood warnings is always agitated after every serious flood, and government and state authorities are importuned to do something, or to do more than they have done.

The United States Weather Bureau has for many years had certain flood stations in New England, which, by telephone or telegraph, report to the central authority, and from there news of floods is given to the press and the public. Generally speaking, this service has been good, but there are several reasons why it can never be entirely satisfactory. On most of the tributary streams the time element is entirely too short for this type of warning to be of much assistance, particularly as few of the Weather Bureau stations maintain 24-hour attendance. Furthermore, special knowledge of the behavior of a river under flood conditions is necessary for the correct interpretation of the data as it comes in.

On a large river like the Connecticut or Merrimack there is time for widespread publicity and even for warnings in the press. The latter often fail in effectiveness, however, because of frequent false alarms in the past in the effort to make striking headlines which exaggerate what turns out to be an ordinary situation. When the unusual flood occurs the warnings in the press are therefore often disregarded.

On the smaller rivers, as in many instances in the flood of 1927, less than 24 hours may be available in which to issue warnings, and the agency must be a local one to be effective. The conditions existing in 1927 put out of commission the telephones, electric lights, railroad and trolley communications, flooded the highways, and made automobile and horse-drawn traffic difficult. With highway and railroad bridges washed out or made dangerous, some entire river valleys were paralyzed

with fear, and if it had not been for outside help from more fortunate communities not affected, the loss of life and property in Vermont would have been greater.

It will be difficult for any one to fully realize the conditions obtaining in Waterbury during the flood of 1927 without having seen them.

After a very heavy 48-hour rain the situation began to look very serious at about four o'clock on Thursday afternoon, November 3. At six the fire alarm was sounded. The water was then about 3 feet deep on Randall Street. Even then the seriousness of the situation was not realized.

Some people left their homes, but most had to be brought out in a boat and a canoe, the only available craft for rescue work.

For about 3 hours the water rose at about the rate of 3 feet an hour, and from then until about four o'clock in the morning, Friday, at about 1 foot an hour.

Fully three-quarters of the village, other than the central business portion, on a slight eminence of only 18 feet, which divides the village, was inundated. First floors were entirely covered, and in a great many homes the water swirled above the level of the second floor. More than 20 houses were carried away or tipped over.*

At Bolton . . . the Winooski . . . within a few hours wore away the railroad embankment, rushed through and filled the valley clear across with a wild sweeping current. Caught helplessly in their habitations about 26 persons, swept along as these floated away, perished under appalling circumstances.†

Rate of Flood Rise

In Table 31 are given approximate data of time of occurrence of flood peaks from the beginning of the storm, as well as rate of rise in river stage for various streams where great damage and in many cases loss of life occurred.

The most rapid average rate, based upon total rise and elapsed time, is 2.07 feet per hour for the Winooski River at Montpelier Junction, Vt. The maximum hourly rate of 2.60 feet per hour occurred on the Westfield River, at Westfield, Mass. It is said that at Waterbury and Bolton, Vt., a rise occurred of from 3 to 4 feet per hour, or about a foot every 15 minutes, a rise too rapid to permit individuals to think calmly and plan adequately to save their families and property.

* From p. 122 of "The Vermonter," Vol. 32, No. 8.

† From p. 91 of "Vermont in Floodtime."

TABLE 31. — RATE OF RISE FOR CERTAIN STREAMS DURING FLOOD OF NOVEMBER 3-4, 1927.

Location	Drainage Area (Square Miles)	Time of Beginning	Time of Peak	Elapsed Time (Hours)	Total Rise (Feet)	Average Rate of Rise (Feet per Hour)
Winooski River, at Montpelier Junction, Vt.	418	Nov. 3, 6 A.M.	Nov. 3, 9 P.M.	15	31.0	2.07
Winooski River, at Waterbury, Vt.	724	Nov. 3, 6 A.M.	Nov. 4, 4 A.M.	22	34.0	1.55
Winooski River, at Gorge above Winooski, Vt.	1,060	Nov. 3, 6 A.M.	Nov. 4, 3 P.M.	33	52.0	1.58
Lamoille River, at Fairfax Falls, Vt.	529	Nov. 3, 6 A.M.	Nov. 4, 4 P.M.	34	18.0	.53
Westfield River, at Westfield, Mass.	496	Nov. 3, 4 P.M.	Nov. 4, 8 A.M.	16	20.0	{ 1.25 2.60*
Connecticut River, at White River Junction, Vt.	4,120	Nov. 3, 12 M.	Nov. 4, 6 A.M.	18	30.0	1.67
Connecticut River, at Sunderland, Mass.	8,000	Nov. 3, 10 A.M.	Nov. 5, 4 P.M.	54	27.0	.50
Middle Branch at Goss Heights, Mass.	53	Nov. 3, 8 A.M.	Nov. 3, 4 P.M.	8	6.5	.81
Deerfield River, at Charlemont, Mass.	180	Nov. 3, 9 A.M.	Nov. 3, 9 P.M.	12	10.0	.83
Pemigewasset River, at Plymouth, N. H.	615	Nov. 3, 10 A.M.	Nov. 4, 5 P.M.	31	25.0	{ 1.50* .81 2.00*
Merrimack River, at Franklin Junction, N. H.	{ 1,460	Nov. 3, 8 P.M.	Nov. 5, 3 A.M.	31	25.0	.81
Merrimack River, at Lowell, Mass.	4,100	Nov. 3, 12 P.M.	Nov. 6, 4 A.M.	52	15.0	{ 1.60* .29 .67*
Miami River, at Hamilton, Ohio	—	Mar. 23, 1913, 12 P.M.	Mar. 28, 1913, 3 A.M.	51	35.0	{ .69 2.00*

* Maximum rate of rise, taken from flood hydrograph.

A study of Table 31 shows a relation between the time from the beginning of the storm to the arrival of the flood peak at any point, and the length of the drainage area above that point. Taking the length as the square root of the drainage area, and dividing the time required to reach the peak by this length, gives the following times in hours per mile of length: *

Average, 1.1.

Minimum, .7.

Maximum, 1.3.

This time in hours between the beginning of rainfall and the arrival of the flood peak (which is the concentration period) may therefore be expressed approximately as, $t = \sqrt{A}$, where t is time in hours and A is drainage area in square miles. The variation for different streams appears to be about 33 per cent either way from this result.

This relation is for the usual river in New England, and shows what time is commonly available, in part, at least, for flood warnings between the beginning of the storm and the flood peak, varying from a few hours for small streams to a day or more for large rivers.

Actually, some time will elapse after a storm begins before it becomes evident that it is of sufficient intensity to cause a flood. Moreover, flood damage and loss of life may occur before the flood peak is reached. The time available for flood warnings may therefore be materially less than that given in the equation $t = \sqrt{A}$, probably not over half that time in many cases.

Agencies for Flood Warnings

United States Weather Bureau. — For public weather information and flood warnings, the United States Weather Bureau is the central agency. The daily weather chart and forecast of the United States Weather Bureau is now generally available, giving records of temperature and precipitation, the areas of high and low pressures, and weather expected for the next day. From this map it is possible to anticipate likely weather conditions, and to follow the path of any great storm, particularly the tropical storms, which may cause excessive rainfall in New England. The daily forecast of the United States Weather Bureau should contain not only the precipitation for the last 24 hours, but also the accumulated precipitation from the beginning of the storm, and a flood warning in large type placed on this bulletin when warranted by

* This is excluding data for the two Connecticut River stations (3.6 at White River Junction and 1.7 at Sunderland), because of the effect of the White River in causing a peak before flow from the full drainage area had time to concentrate.

the facts. Depth of snow generally prevailing should also be noted at times when this is likely to be an important factor in flood flow.

The United States Weather Bureau should have more flood stations and have their warnings broadcast by radio as well as in the press. The statement attributed to Charles F. Marvin, Chief of the United States Weather Bureau, that additional Weather Bureau stations are to be established along all official airways, suggests that possibly flood records and warnings may be included as a part of this work.

Local Records. — Every village or town in New England located on a river, tributary stream, or in a river valley should have at least one rain gage and some central authority — as selectmen, postmaster, water superintendent — to whom warnings may be sent from other points.

Flood marks of previous great floods should be left in conspicuous places, and kept in mind by the railroad and highway officials, city and town engineers, postmasters and selectmen.

For each city and town affected, flood maps should be made of the 1927 freshet or greatest previous flood, showing the areas flooded and surrounding high land.

The engineering departments of the states, counties and cities, through their engineers or officials, should have office records of floods and flood marks, and be in a position to locally interpret flood warnings.

Public Service Companies. — At freshet times every public service or private company which generates power by water should have information regarding the existing snowfall, rainfall and river stage heights on their river.

Such companies with the use of the radio, the telephone and telegraph, and in co-operation with the United States Weather Bureau, are the best medium for issuing flood warnings when, and only when, they are needed. No single authority or set of rules will ever take the place of the constant supervision by operating men who know the rivers. The public service companies should, however, definitely recognize and assume their responsibility to the public in this respect, and co-operate with one another, and arrange for the adequate and immediate publication of the necessary flood warnings.

Chapter VIII. — Cost and Economic Phases

In this report there have been discussed the features of the New England flood of November, 1927, its comparison with earlier floods, flood probabilities, and measures which may be taken to prevent or mitigate the effect of future floods. It remains, however, to draw a distinction between what is possible and what is profitable; what good judgment will warrant in the way of expenditures which may not be needed for many years, if ever, and what is warranted where flood protection may be combined with other benefits which may accrue to a given community or river district.

The losses in New England directly traceable to the flood of November, 1927, are given as about \$40,000,000, of which nearly two-thirds was in Vermont, where almost all the loss of life occurred.

The question arises, How far will this occasional damage warrant the expenditures of large sums of money for flood prevention and flood protection? In other words, have we a flood problem in New England which warrants extensive and costly flood protection works such as were constructed on the Miami River? The answer is, probably, no. It is doubtful whether any extensive plan to protect large areas is economically warranted on the basis of present flood history, but there may be places where some measure of complete or partial control at strategic points may be warranted even at present. While the cost of preventive or protective structures may be spread over a period of years by bond issues and annual payments, these improvements are not warranted unless there are larger and more frequent losses than in the past. If damaging floods occur more than a generation apart, they are soon forgotten, and when they do come are looked upon as fate and taken as such. However, some improvements may be necessary in some cases where floods become a constant source of economic loss which cannot be tolerated.

From a study of the few flood prevention projects in this country and abroad which have been carried out successfully, the storage of large amounts of water in artificial reservoirs or in natural lakes has been found to be the best single element of flood control. In some cases water storage for the complete or partial control of a certain portion of the drainage area, combined with construction of dikes or improved channels for local protection at certain points, has been the solution.

Such a combination of reservoirs and channel improvements, built at a total cost of \$28,000,000, or about \$7,000 per square mile, exists

today for the satisfactory control of 4,000 square miles of the Miami River and tributaries, and protection of Dayton and other communities situated in the Miami valley, warranted by a flood damage of about \$100,000,000 in 1913.

Where large storage exists, as on the upper branches of most of the large rivers of Maine, the flood problem is usually unimportant; yet on the lower reaches of the same rivers there may be sufficient areas of uncontrolled drainage to make the flood problem a difficult one. The Kennebec River drainage area above Waterville, Me., is a good example of this unbalanced condition, which is gradually being taken care of through the building of new dams and reservoirs on the main river and tributaries.

For whatever purpose, the cost of reservoirs for water storage runs all the way from a few dollars to several thousands per million cubic feet. One million cubic feet is equivalent to 11.6 cubic feet per second for 1 day, an absurdly small amount of storage considered from the flood point of view. Table 32 gives the cost of a large number of the storage reservoirs, and the purpose for which they have been built.

TABLE 32. — STORAGE RESERVOIR COSTS

LOCATION	Total Cost	Capacity (M. C. F.)	Cost per M. C. F.	Remarks
<i>New England</i>				
Pawtuxet River, Scituate	\$6,920,000	5,000	\$1,380	Water supply
Nashua River, Wachusett	11,000,000	8,500	1,294	Water supply
Deerfield River, Davis Bridge	4,500,000	5,000	900	Power and storage; about \$2,000,-000 chargeable to storage
Westfield Little River, Cobble Mountain	2,225,000	2,980	755	Water supply and power (under construction; cost estimated)
Deerfield River, Somerset	1,000,000	2,300	436	Water power
Androscoggin River, Azischohos	1,000,000	9,500	105	Water power
<i>American and Canadian</i>				
Scioto River, Columbus, Ohio	2,220,080	715	3,110	Water supply
Miami River, Huffman Dam	5,809,158	7,300	795*	Flood protection
Escondido, Cal.	110,059	152	723	Irrigation
Ashokan Reservoir, N. Y.	12,670,000	17,650	718	Water supply
Pecos River, Texas	176,000	274	642	Irrigation
Allegheny and Monongahela Rivers (16 reservoirs)	12,424,110	20,365	610	Flood protection and water power
Cache le Poudre, Col.	125,000	246	505	Irrigation
Miami River, Lockington	1,324,783	3,050	434*	Flood protection
Miami River, Taylorsville	3,486,894	8,100	430*	Flood protection
Sacandaga Reservoir, N. Y.	9,000,000	30,000	300	Water power
Miami River, Germantown	1,378,859	4,630	297*	Flood protection
Miami River, Englewood	3,792,775	13,600	279*	Flood protection
Larimer and Weld, Col.	90,000	325	276	Irrigation
Sweetwater, Cal.	264,074	985	269	Flood protection
Stillwater Reservoir, N. Y.	1,000,000	4,500	222	Water power
Calaveras River, Cal.	1,500,000	7,100	210	Flood protection
Chippewa and Flambeau Rivers, Wis. (several reservoirs)	2,000,000	17,000	118	Water power

Cuyamaca, Cal.	56,400	497	113	Irrigation
Lake McMillan, Texas	200,000	2,875	52	Irrigation
Elephante Butte Dam, Ariz.	5,866,000	114,500	51	Flood protection
Bear Valley, Cal.	75,000	1,740	43	Water power
Indian Lake, N. Y.	100,000	4,700	21	Water power
Mississippi (6 reservoirs in Minnesota)	1,761,228	97,846	18	Navigation
St. Maurice River, Quebec	2,500,000	160,000	16	Water power
Ottawa River, Quebec	840,000	168,000	5	Water power (several reservoirs)
Tyler, Texas	1,140	1,146	1	Irrigation
<i>Foreign Countries</i>				
Wien River, Vienna	1,700,000	56.5	30,000	Flood protection
Elbe and Oder Rivers, Austria	3,520,000	800		Flood protection and power
Ruhr River, Henne Reservoir	805,000	388	2,070	Water power
Oder River, Germany	1,458,000	780	1,870	Flood protection
Rhine River, Germany	12,750,000	8,500	1,500	Flood protection and power
Bober River, Germany	1,790,000	1,765	1,015	60 per cent for floods, 40 per cent for power
Weser River, Germany	4,510,000	7,145	632	Canal feeder
Rur River, Germany	1,000,000	1,606	620	Flood protection
River Nile, Assouan Dam	24,360,000	81,200	300	Irrigation
<i>United States Reclamation Service Reservoirs in Mississippi Valley</i>				
Pathfinder	2,000,000	43,630	46	—
Lake Mintare	500,000	2,645	189	—
Lake Alice	100,000	523	191	—
Winter Creek	100,000	161	621	—
Belle Fourche	1,200,000	8,830	136	—
Willow Creek	200,000	740	270	—
Shoshone	1,400,000	19,900	70	—
Nelson	200,000	1,741	115	—

* Construction cost, exclusive of real estate damages, administration, and general expense, which average \$160 per m. c. f. additional.

TABLE 32. — STORAGE RESERVOIR COSTS — *Concluded**Proposed Reservoirs*

LOCATION	Total Cost.	Capacity (M. C. F.)	Cost per M. C. F.	Remarks
<i>Vermont *</i>				
Missisquoi River (7 projects)	\$8,927,000	6,900	\$1,300	—
Passumpsic River (5 projects)	3,770	3,770	1,000	—
Lamoille River (11 projects)	9,271,000	9,660	960	—
White River (6 projects)	6,814,000	8,200	830	—
Winooski River (8 projects)	9,350,000	14,500	645	—
<i>Mississippi Flood Commission</i>				
On St. Francis, White and Arkansas Rivers	215,000,000	1,089,500	198	Flood protection
Above Cairo, Ill.	285,000,000	1,693,000	185	Flood protection
<i>Miscellaneous</i>				
Colorado River, Boulder Dam	50,000,000	100,000	500	Irrigation and power
Missouri River (5 reservoirs)	1,750,000	14,000	125	Irrigation and power

* Reservoirs proposed in connection with power developments; figures given are for storage alone.

What can the community or district afford to pay for flood protection or for insurance against losses?

Reservoir capacity of 4 inches depth over a drainage area (or 9.3 million cubic feet per square mile) would make a 6-inch flood practically harmless, and it would keep an 8-inch flood down to a point where little damage would occur.

The cost of a storage reservoir in New England, except under unusually favorable conditions, is likely to be from \$600 to \$1,000 per million cubic feet of capacity, or \$5,580 to \$9,300 per square mile, to give complete flood protection by reservoirs.

The flood of 1927 in Vermont caused a total damage of about \$30,000,000, chiefly distributed over about 5,000 square miles, or about \$6,000 per square mile. If a similar flood is assumed to occur once in 50 years, the average annual loss would be \$600,000, or \$120 per square mile. The maximum sum which can be justified for flood prevention is the capitalized value of the average annual loss. If the rate of interest is taken as $4\frac{1}{2}$ per cent, an expenditure of over \$2,670 per square mile would not be warranted on the assumptions as stated above.

The above figures assume complete regulation. There are some cases where partial control may result in a considerable reduction of flood damage, and the cost would be proportionately less compared with the value of the results obtained.

If a reservoir can be created for the benefit of all on the river and for water supply, water power, or flood control purposes, even at great cost, it may be warranted by the results accomplished. When a community has been safely through several great freshets of a hundred years or more, it is likely that the construction of one large reservoir may conserve just enough of the drainage area to make that particular river valley safe against the unusual flood. A case in point is the Kennebec River in Maine, which appears to lead most of the great rivers of New England in flood run-off per square mile.

The State of Vermont has been investigating the possibility of flood control incident to power storage reservoirs, and is working out a plan in co-operation with the power companies for the protection of all the principal rivers of the state.

There are still in New Hampshire certain streams, tributary to the upper Connecticut and Merrimack Rivers, which could be controlled by the building of reservoirs. These improvements are likely to come in time. The Fifteen Mile Falls power development on the upper Connecticut will probably have some effect on the lower Connecticut River, but the river is so long and has so many large tributary streams

that an economical plan of controlling anything like 25 per cent of the drainage area seems, at the time of writing, to be almost out of the question.

The most desirable policy toward the flood problem here in New England appears to be to encourage in every way the construction of storage reservoirs for power purposes, as a by-product of which a large measure of flood protection may be afforded at little or no added cost. Storage above spillway level under competent operation will, even with full reservoir, greatly cut down flood peaks. Furthermore, under normal conditions the power reservoir is drawn down the greater part of the year, and thus normally functions to lessen floods. As our rivers become more completely developed as to storage and power, not only will their flow be well sustained in dry periods, but flood tendencies will become less and less.

We need additional legislation to facilitate a complete program of storage. This legislation should permit the formation of storage regulating districts and provide the right of condemning land for water power storage purposes, as well as provisions for assessing the costs upon those benefited. Such laws are in operation in New York, Kansas and, to a limited extent, Wisconsin. The states might well afford to finance all or part of the cost of such reservoirs, giving the projects the benefit of a low rate of interest. They might also assist in doing the necessary work of making the changes in roads incident to reservoir construction as a part of their general highway work. This is all a long-time program which can only keep in step with power demand. As this continues to increase, more and more reservoir sites may be developed, and in time most of the flood hazard may be reduced to a point where it is not serious.

There will also be special cases to be dealt with where other methods must be employed. Comprehensively, however, the general solution of the flood problem here in New England appears to be the development of the power reservoir systems encouraged by helpful legislation.

OF GENERAL INTEREST

BOSTON ENGINEERS AWARDED PRIZES FOR DESIGN OF LAKE CHAMPLAIN BRIDGE

The firm of Fay, Spofford & Thorn-dike, Engineers, of Boston have won signal honors for the design of the Lake Champlain Bridge, which connects Crown Point, New York, to Chimney Point in Addison, Vermont, which was completed last year. They have been notified that the American Institute of Steel Construction has, through its jury of award, given them honorable mention because it "felt that the structure was of outstanding merit." A letter from that association says "this honorable mention for the Lake Champlain Bridge was the only other award (besides the first prize) made on this class of bridge; that is, over \$200,000 in cost." This was the second annual competition for the most æsthetic solu-

tions of steel bridge construction in the United States and Canada.

On July 9 Mr. Spofford, one of the partners, attended the annual convention of the American Society of Civil Engineers at Cleveland, where he received a silver medallion representing the second prize of the Phebe Hobson Fowler Architectural Award "for a work of outstanding merit in the architectural design of a bridge." Designs for which the Fowler Award are made "must be fundamentally artistic, complying with the principles of Simplicity, Symmetry, Harmony and Proportion, and not merely a structure decorated with Classic, Renaissance, Romanesque, or other architectural details or trimmings."

PROCEEDINGS OF THE SOCIETY

JOINT OUTING, NEW ENG- LAND WATER WORKS AS- SOCIATION AND BOSTON SOCIETY OF CIVIL EN- GINEERS

The regular meeting of the Boston Society of Civil Engineers was omitted in June in order to join with the New England Water Works Association in an outing which was held at the Hotel Pilgrim, Plymouth, on Wednesday, June 25, 1930. A very enjoyable program was arranged to include sports, bathing and visits to

places of historical interest, and to see the manufacture of cement pipe. About 150 members, ladies and guests attended.

APPLICATIONS FOR MEMBERSHIP

[September 20, 1930]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

CAMP, THOMAS RINGGOLD, Watertown, Mass. (Age 34, b. San Antonio, Texas.) Graduate of A. and M. College of Texas, with degree of B.S. in architectural engineering. Experience: 1916-17, draftsman with Texas Power and Light Company; 1917, September to December, draftsman in construction of Camp Travis, Texas; December, 1917, to April, 1919, in U. S. Army, A. E. F.; May, 1919, to February, 1920, valuation engineer on cottonseed oil mills; February, 1920, to June, 1922, in Breckenridge, Texas, as field engineer, flood protection; assistant to resident engineer in a \$3,500,000 highway project; resident engineer, Hawley & Sands, sewerage system and disposal plant; 1922-23, city engineer, Breckenridge, Texas; October, 1923, to June, 1925, graduate student at Massachusetts Institute of Technology, with S.M. in civil engineering; 1925-28, member of firm, Spoon, Lewis & Camp, consulting engineers of Greensboro, N. C.; November, 1928-29, chief designing engineer for Alexander Potter, New York City; 1929, to date, associate professor of sanitary

engineering, Massachusetts Institute of Technology. Refers to *J. B. Babcock, W. A. Liddell, K. C. Reynolds, G. E. Russell, C. M. Spofford.*

GINSBURG, JACOB, East Boston, Mass. (Age 19, b. East Boston, Mass.) Graduate of Harvard Engineering School, 1930, as a civil engineer. Worked as transitman during summer of 1929 for Massachusetts Department of Public Works. Refers to *R. W. Coburn, Nathan Ginsburg, L. J. Johnson, T. R. Kendall, H. M. Turner.*

HORTON, DONALD FRANCIS, Medford, Mass. (Age 24, b. Haverhill, Mass.) Graduate of Massachusetts Institute of Technology in June, 1927, with degree of S.B. From July, 1927, to June, 1928, was with the Mississippi River Commission as surveyor; June, 1928, to June, 1930, junior engineer, United States Engineer Department, Cape Cod Canal; June, 1930, to date, junior engineer, United States Engineer Office, 13th floor, Custom House. Refers to *J. B. Babcock, G. L. Hosmer, K. C. Reynolds, C. M. Spofford.*

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JAMES M. BETTON, . . .	May	29, 1930
JAMES RICE, . . .	May	29, 1930
ELLIOTT HOWES GAGE, . . .	July	13, 1930
ALLEN HAZEN, . . .	July	26, 1930
IRA N. HOLLIS . . .	August 14,	1930

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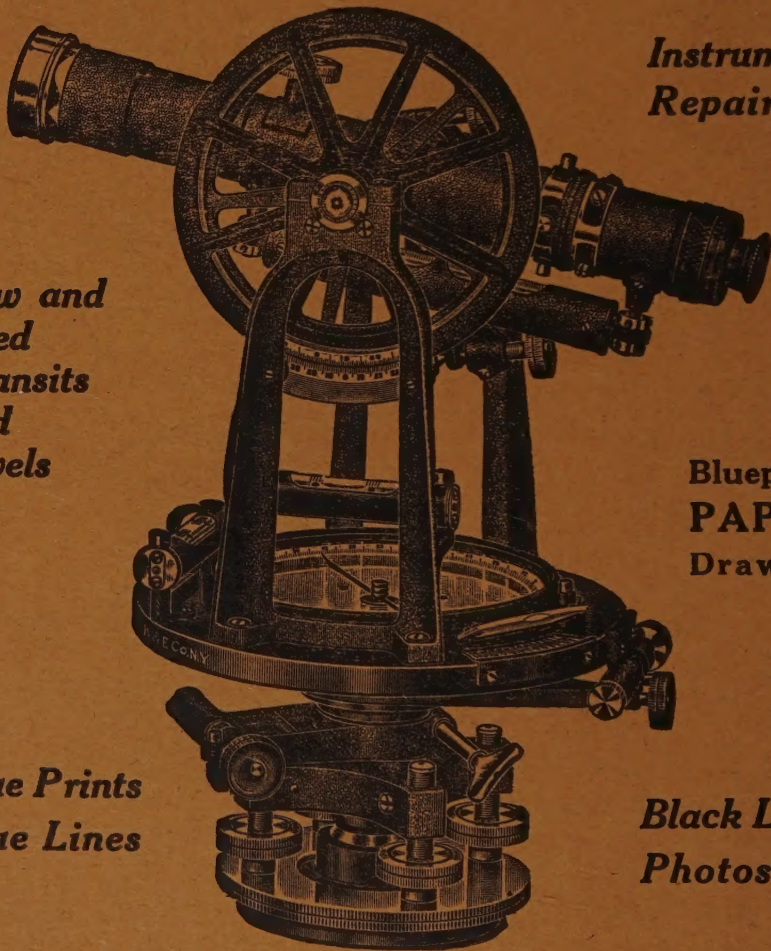
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